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Engineering News-Record

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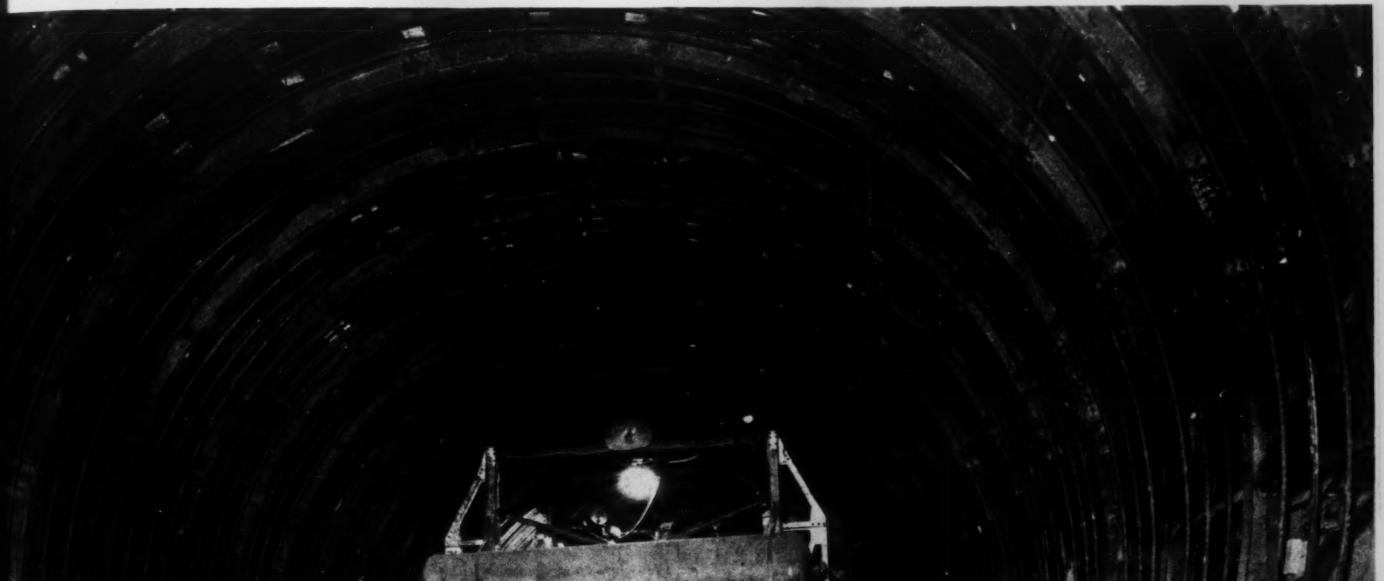

FOUR TUNNELS now being driven for river diversion and outlet works at Fort Peck Dam will have a finished diameter of 24 ft. 8 in. inside the 3-ft. thick lining of heavily reinforced concrete. This picture shows the arch-lining gantry spanning the two service tracks.

Asbestos Bonding

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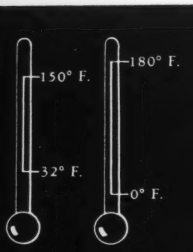

Besides assuring permanently tight adhesion *Asbestos Bonding* actually widens the plastic range of the bituminous coating and pavement—as much as 50 per cent.

WHEN Armco first introduced Paved Invert Pipe with *Asbestos Bonding*, municipal engineers immediately recognized its possibilities. As a result, scores of installations are now in progress or planned for sewer and drainage purposes.

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A MOMENT WITH THE PUBLISHERS

An Engineer *Sells* His Job

THE ENGINEERING PROFESSION was fortunate in its spokesman at the Triborough Bridge opening. When O. H. Ammann, Chief Engineer, responded to the well-deserved tribute of Commissioner Moses, he made admirable use of the opportunity to restate some construction fundamentals. One of these deserves to be emphasized because so many of our people fail to see the construction enterprise and the construction industry in their full stature.

> *"The project,"* said Mr. Ammann, *"is notable as a product of cooperative effort and widespread employment. In its planning the Triborough staff, assisted by eminent consultants, had the active collaboration of eighteen city departments, state agencies and public utility corporations.*
>
> *"Six hundred manufacturing plants distributed through twenty states have furnished their products. Granite from as far north as Maine; New York, New Jersey and Pennsylvania furnished sand, stone and cement for the 2,000,000 cubic yards of concrete. Ninety-one thousand tons of steel and iron were produced in the mills and shops of thirteen states from Rhode Island to Indiana and south to Alabama.*
>
> *"For two and one-half years these products moved to the site by rail, water and highway. Under seventy-four contracts the construction work at the site was performed by over two hundred contractors and subcontractors, who at times employed as many as 3,000 workmen. Much of the success is due to their splendid efforts and cooperation.*
>
> *"On behalf of the engineering staff, I wish to express our thanks to all those who have helped us and our appreciation of the opportunity to contribute toward this great public work under the inspiring leadership of Mr. Robert Moses."*

IT IS noteworthy that in this generous acknowledgment Mr. Ammann recognizes not only the professional services of his staff and consultants but also the essential services rendered by other departments and utilities and by the contractors and subcontractors.

But his vision ranges even beyond those whose skill and energy were expended at the site of the bridge to include also those unidentified thousands whose genius and labor found equally direct expression on the bridge even though their work was done hundreds of miles distant and through many callings.

It would be helpful if all could understand the degree to which construction actually is an assembly operation. It brings to a prepared site a variety of materials and appliances in crude, semi-finished and finished form; it assembles also, for the purpose of processing and combining them into a structure, a more or less imposing and costly plant, depending on the magnitude and complexity of the job. This plant is none the less comparable with that of many factories because it happens to be mobile and to consist of separate elements that may be dissociated at the completion of the job and moved elsewhere to form new combinations.

Most of the units of that plant, most of the materials and appliances built into the structure must be produced and processed elsewhere; that production and processing, the transportation and assembly of plant, materials and appliances is an important part of every job.

IT WILL help if more engineers follow the example of Mr. Ammann in seeing their profession as an integral part of a great industry and if they will take every opportunity to sell their jobs, not only on the basis of statistics of accomplishment but also on that of productive employment provided for labor and capital.

Too many American families now consider an engineering construction project merely as a technical problem for the engineers and a business opportunity for the contractors. If only they could be made to see that it may involve also the factory where father works, the mill or power-house that employs brother Jack and the insurance office where sister Maggie thumps her typewriter, the construction industry would be making real headway with its job of selling the American people a better understanding of its influence on their welfare.

Willard Chevalier

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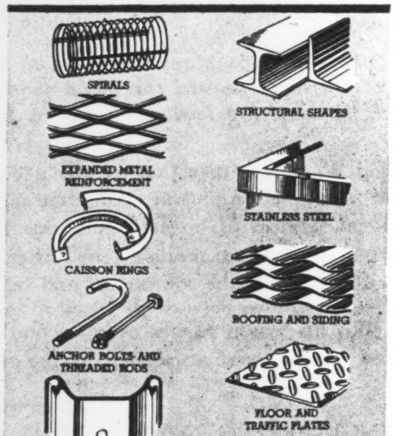

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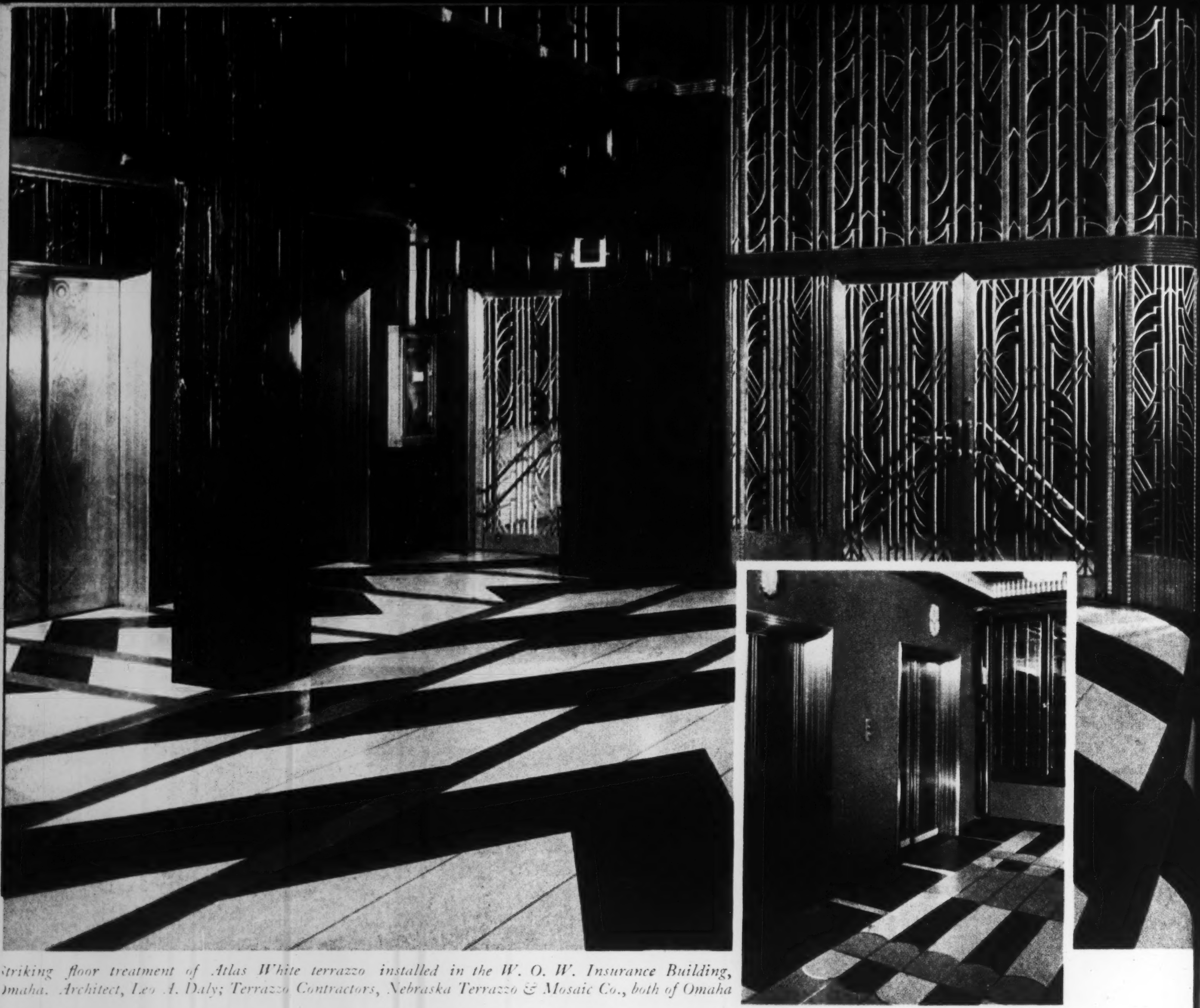
Another Bay Bridge Article—AS THE GREAT BRIDGE ACROSS SAN FRANCISCO BAY HAS PROGRESSED, ENGINEERING NEWS-RECORD HAS REPORTED CURRENTLY ON THE WORK IN A SERIES OF NOTABLE ARTICLES BY THE THREE PRINCIPAL ENGINEERS ON THE JOB. ANOTHER ARTICLE OF THE SERIES—ONE ON SUSPENDED STRUCTURE—WILL BE PUBLISHED NEXT WEEK.



Striking floor treatment of Atlas White terrazzo installed in the W. O. W. Insurance Building, Omaha. Architect, Leo A. Daly; Terrazzo Contractors, Nebraska Terrazzo & Mosaic Co., both of Omaha

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ENGINEERING NEWS-RECORD

August 13, 1936

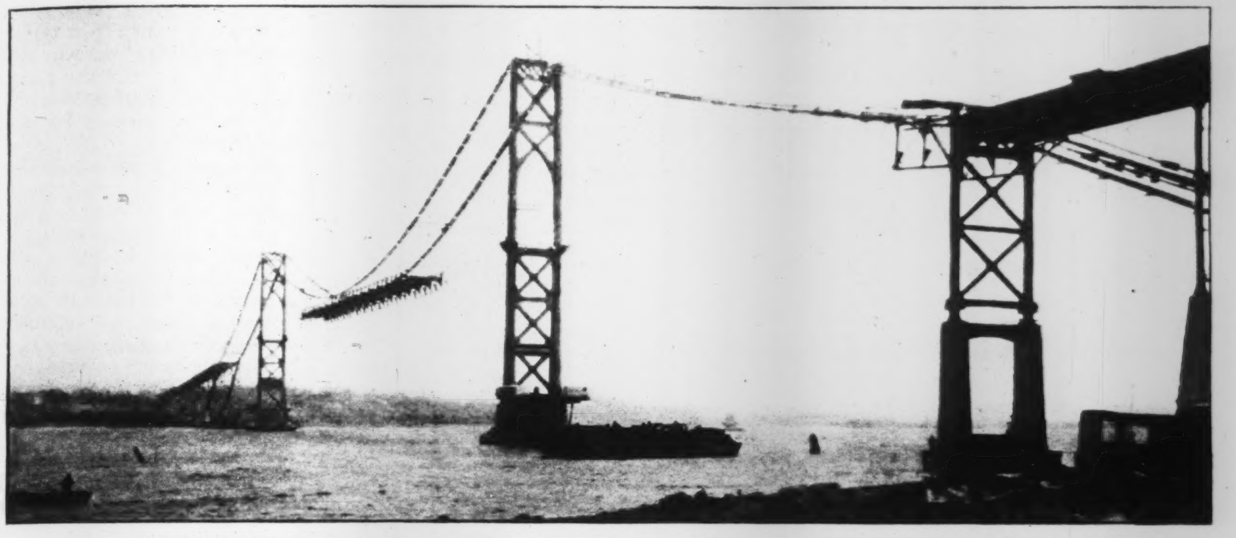


FIG. 1—CONSTRUCTION VIEW of Mt. Hope bridge whose initial heat-treated wire cables developed breaks that led to dismantling of the bridge and replacement of the cables with cold drawn wire. The main span is 1,200 ft., the side spans 504 ft. and the backstays 220 ft.

Heat-Treated Bridge Wire Failures Reproduced in Tests

Investigation by Bureau of Standards of wire removed from Mt. Hope Bridge indicates inability of crystalline microstructure to resist fatigue fracture under fluctuating tensile loads as the cause of failure

THE UNSUCCESSFUL attempt to introduce heat-treated wire into suspension bridge construction, which was marked by the failure in 1929 of a large number of wires in the cables of the Mt. Hope Bridge in Rhode Island and the Ambassador Bridge at Detroit, followed by the dismantling of the bridges before they were finished and their subsequent reconstruction with cold-drawn wire cables, is familiar to most readers of this journal and may be reviewed in the issues of April 11, 1929, p. 602, and Oct. 10, 1929, p. 562. Ready explanation of the failures was lacking, even from metallurgists, and the Bureau of Standards was prevailed upon to undertake a study of the problem. A six-year investigation followed, and the results became available for the first time in a paper presented at the recent American Society for Testing Materials convention (*ENR,* July 9, 1936, p. 58) by W. H. Swanger and G. F. Wohlgemuth of the bureau staff. Only the wire from the Mt. Hope Bridge was studied, since the Ambassador Bridge wire was identical, having been manufactured at the same time in the same mill.

In brief, the conclusions point to inherent characteristics of the heat-treated wire and consequent special erection practices, as the basic cause of the failures. The wire being stiff and having a high elastic limit required preforming, to fit the curve of the anchor shoe. This curvature was not always accurate, however, and high elastic bending stresses were set up with which were combined the fluctuating tensile stresses during the erection period. Furthermore the bending stresses were highest for minor changes in tensile stress and at low values of tensile stress. Surface imperfections provided points for localizing these stresses, with consequent fatigue cracks. The crystalline microstructure of the wire, as contrasted with the fibrous structure characteristic of cold drawn wire, offered little resistance to the spread of these cracks, and failure resulted.

The unsatisfactory wire was made of basic open hearth steel, rolled into 4-in. square billets, reheated and rolled into 0.225-in. rods, then cold drawn to 0.192 in., and heat-treated to desired strength. Its chemical composition was the same as conventional cold-drawn wire, which in existing large bridges is almost universally acid open hearth steel, cold drawn from heat-treated or "patented" $\frac{3}{8}$-in. rod to 0.192-in. wire, this cold-drawing operation inducing the required strength. Both varieties are cleaned in acid and galvanized for rust protection by running through molten zinc. Heat-treatment and galvanizing of the unsatisfactory wire was done continuously by pulling the wire endwise through a series of tanks (Table I). Representative properties of the heat-treated wire and the cold-drawn wire which replaced it are shown in Table II. The microstructures are compared in Fig. 2.

TABLE I—SEQUENCE OF HEAT TREATMENT OPERATIONS

Tank	Medium	Temperature °F.	Seconds Immersion
Preheating pan	Lead	1325 to 1350	24
First hot pan	Lead	1550	35
Hot quench	Lead	1250	33
Second hot pan	Lead	1550	37
Cleaner	Fused salts	(?)	5
Cold quench	Oil	225 to 275	90
Sand box	Sand	200(?)	10
Tempering bath	Lead	810 to 850	18
Pickle	Dilute HCl	150	75
Wash	Water	(?)	
Flux	$ZnCl_2$ & NH_4Cl	800(?)	
Galvanizing pan	Zinc	850	35

TABLE II—PROPERTIES OF TWO TYPES OF BRIDGE WIRE

	Heat Treated Wire		Acceptance Specification	Cold Drawn Wire	
	Before Heat Treatment	After Galvanizing		Before Galvanizing	After Galvanizing
Ultimate lb. per sq.in.	153,000	223,000	220,000	247,000	223,000
Yield (Stress at 0.75% elongation)	......	189,800	190,000	......	182,600
Proportional limit	......	99,700	......	......	84,200
Elongation, 2 in. per cent.	2	6	4	3	6
Red. of area, per cent.	20	38	30	41	40
Hardness, Rockwell C.	27	46	..	40	41

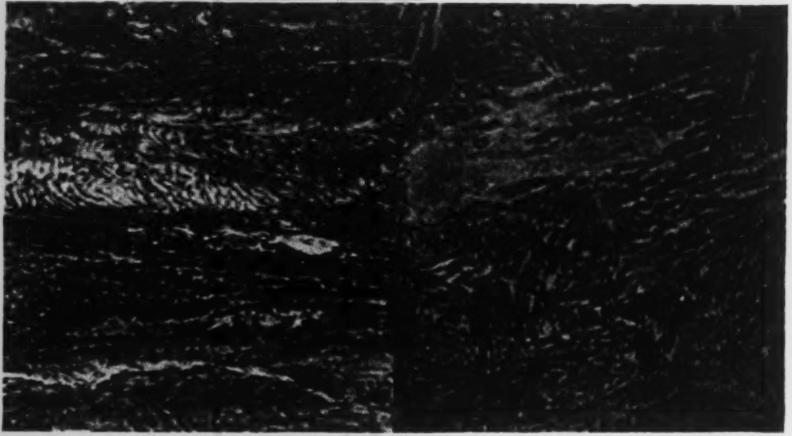
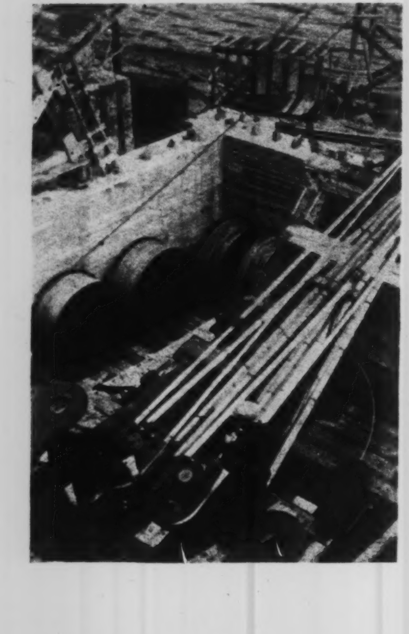


FIG. 2—FIBROUS MICROSTRUCTURE of cold drawn wire (left) compared with crystalline structure of heat-treated wire (right).

FIG. 3—SPINNING was special in that the stiff heat-treated wire was preformed to the curve of the anchor shoe by passing it around small diameter sheaves, shown near the lower edge of the shoe.



The first step in the investigation of the failures was the segregation of the possible contributing factors and the elimination of those that were improbable. Thus it was determined that the wire was in the same condition, structurally, when it failed on the bridges as when it left the mill, and that the use of basic open hearth steel was not significant since no important difference could be found in chemical analysis or inclusions between the heat-treated and cold-drawn wire.

Reviewing the experience in manufacturing the wire, the investigators report that of the 29,224 coils of wire completely processed, some 4,000 were rejected in the testing room or on the processing unit—about half because of defects in the galvanizing and half because of failure to meet the specified physical properties or because they broke in the processing unit. The cause of failure to meet the specified physical properties was in most cases traced to surface defects in the rolled rod or drawn wire. The cause of most of the failures on the processing unit was not ascertained in the wire mill.

Fractures without reduction of area, it was learned, occurred in many coils shortly after they were removed from the heat-treating unit; 337 such breaks were not discovered until the coils were taken from the warehouse, to be reeled for shipment. This condition naturally caused concern and before the wire was wound on the shipping reel it was passed through a roll, straightening machine. Breaks were produced in only 20 more coils by this operation.

Since some of the wire after being run through the heat-treating unit bypassed both acid tanks and the zinc pan and exhibited no breaks when reeling on 30-in. diameter coils, hydrogen embrittlement was thought of as a possible cause of the early breakages in the other coils. However, since hydrogen embrittlement in steel is known to be a transitory effect, disappearing in time, it is stated with confidence in the report that the fractures on the Mt. Hope Bridge cannot be ascribed to this cause; however, many of the mill breakages may be explained by it.

Supplementing these studies by tests, the bureau found no reason to question variability in hardness or torsion resistance between broken and unbroken specimens of wire. Coiling experiments failed to produce any significant breaks.

Preforming loops at shoes

Turning next to possible contributing causes in the erection procedure, the investigators dismissed all operations as unimportant except those required at the anchorages. Since the heat-treated wire was stiff it could not be looped around the anchor shoe, 19¾ in. in diameter, without stressing it beyond the yield point. The wire therefore was preformed at the loop by bending it 1¼ turns around a sheave 9½ in. in diameter (Fig. 3). When released the loop would spring open, because of the high elastic limit of the wire, to the curvature of the circular portion of the anchor shoe. The loop was then placed around the shoe where it theoretically would "lie dead" and be subjected only to tensile stresses.

All of the fractures in the anchorages occurred close to the tangent point on the anchor shoe, and at about the time that the floor steel was being erected. The total number of broken wires was 402, distributed as shown in Fig. 4, and comprising 16 per cent of the wires in the two cables. The nominal static load on the cables at the time the failures occurred produced a tensile stress in the wire of 32,000 lb. per sq.in.

It was concluded that the stress conditions at the anchor shoes were radically different from those in the compacted cables, also that the concentration of breaks in two strands could not be attributed to bad wire since wire from 21 different heats was involved but that the explanation might lie in unusually poor preforming at the two shoes in question or in higher stresses induced by unusual movement of the anchor shoes in adjusting the sag in the completed strands. These facts and suppositions having been established, tests to reproduce the field conditions were undertaken.

Low reduction in area

Comparison of broken wire and wire which had not broken at the shoes showed good agreement in several hundred tension tests, with the exception of some low values for reduction of area, also experienced in the original inspection. In any length of wire, either from the loop around the anchor shoe or from a straight portion, it was found that generally one out of 7 to 10 tensile test specimens fractured with practically no reduction of area, whereas the specified value was 30 per cent. The cause was found to lie in the surface conditions of the wire, for when the outer layers of the galvanized wire were removed mechanically to a depth of 0.015 in., all low reduction breaks were eliminated. The removal of only the shallower defects did not cause any marked improvement, so that the decrease in ductility was evidently caused by the more deeply seated defects, occurring sporadically, but apparently in all of the wire.

An early suggestion was that the essential cause of the fractures might be a slowly creeping tensile load on a wire bent to such a curve that its extreme fiber was stressed beyond the yield point. To try out this hypothesis, specimens of wire about 8 ft. long were subjected to static tensile load while looped over a support similar to an anchor shoe (Fig. 6). Some of the specimens were not preformed, some were very carefully preformed and adjusted while others were improperly preformed. Some were hairpins from the bridge; some had the zinc stripped. Some tests were also made on cold-drawn wire. The initial tensile load was 90,000 lb. per sq.in., and it was increased by 10,000 lb. increments at definite intervals.

Of the 135 specimens of the heat-treated galvanized wire, only 8 broke at tensile stresses less than the nominal ultimate tensile strength, and none at a tensile stress less than 120,000 lb. per sq.in., which is over four times the stress to which the wire in the bridge was subjected. It was concluded that the elastic bending stresses did not have much effect in lowering the effective resistance of the specimens to static tensile loads. Also that static loading under combined stresses was not the mechanism of failure, and that therefore the resistance of the wire to fluctuating stresses should be studied.

Fatigue tests

In a series of fatigue tests, made both by the rotating beam method and also on a Haigh machine where direct tension loads at various ranges were applied, it was notable that machined and polished specimens of the heat-treated and cold-drawn wire acted in substantially the same manner, indicating a fatigue limit of about one-half the ultimate tensile strength. However, similar tests of the failed wire with the original surface intact had a fatigue limit of only 50,000 lb. per sq.in., making it evident that conditions at the surface of the wire caused a drastic reduction in resistance to repeated stresses. With respect to the surface defects, the cold-drawn wire, drawn through five dies from hot rolled rod had a much smoother surface than the heat-treated wire, which had been drawn through only two dies.

The results from the Haigh alternating stress machine tests showed that even though the ultimate tensile strength of each type of wire was 223,000 lb. per sq.in., the maximum range of pulsating tensile stress, at mean stresses from 50,000 to 150,000 lb. per sq.in. was only about 50,000 lb. per sq.in. for the galvanized wire of both types (Fig. 5). Moreover, the heat-treated wire with a much more severely pitted surface than the cold drawn wire had an endurance strength both with and without the zinc coating, approximately as great as that of the cold drawn wire. Large increase

FIG. 4—LOCATION of broken wires at the four anchorages in the seven strands of each cable. Numbers in circles indicate broken wires, those below circles are the strand numbers.

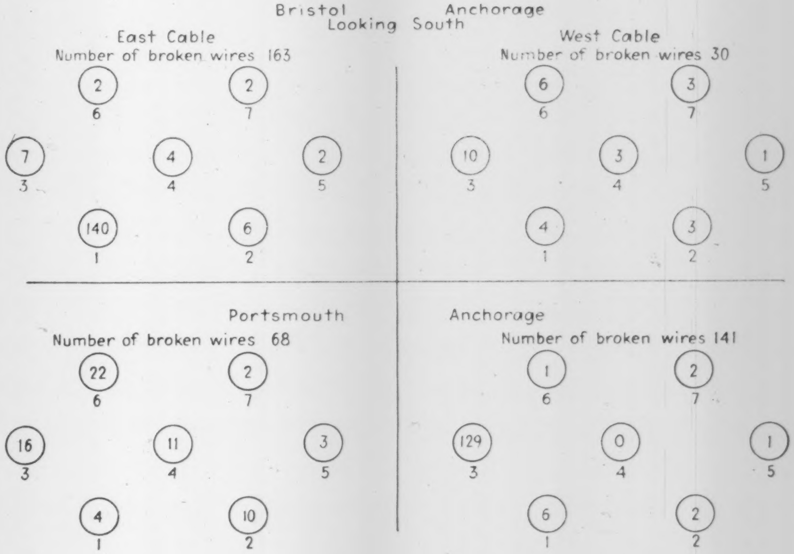
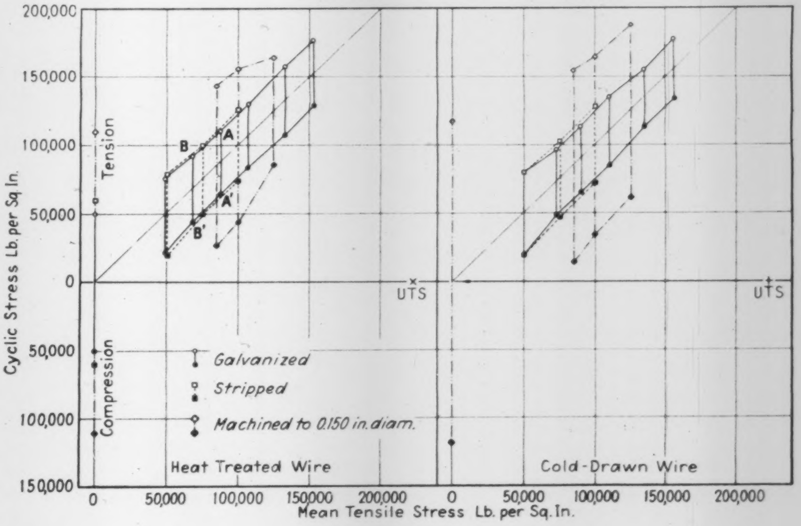


FIG. 5—RANGE of pulsating tensile stress was about same for heat treated and cold drawn wire. Resistance to such stresses determined by range and not by maximum stress. For example range AA' with maximum and minimum of 111,000 and 65,000 has a mean of 88,000; by lowering minimum to 25,000, mean becomes 68,000 for which BB' is safe range.



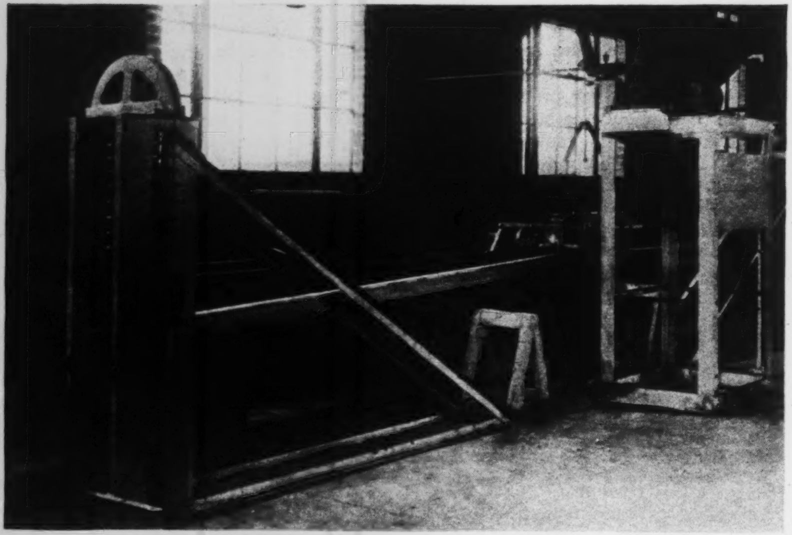


FIG. 6—WIRE TESTING apparatus used by Bureau of Standards for both long-time loading and fluctuating tensile load tests. For the latter the weighted end of the long lever was raised and lowered by a crank. The wire is held to a radius equal to that of the anchor shoe. The horizontal braces on the wire were only used in a special test in which an attempt was made to eliminate herding of the wire.

in fatigue strength obtained by machining away the outer layers is taken as confirmation of the fact that internal cracks or fissures in the heat-treated wire were not a cause of its failure on the bridge. The tests demonstrated that for repeated loadings within the ultimate strength, the important question is not "How heavily is the wire loaded" but "How far is the wire unloaded."

It was evident from the fatigue test that the failures did not occur because the fatigue strength of the heat-treated wire was less than that of the cold-drawn wire which replaced it. Both types of wire were subjected to the same loading conditions — fluctuating tensile load—but it is pointed out, not necessarily to the same stress conditions in the portions looped around the anchor shoes.

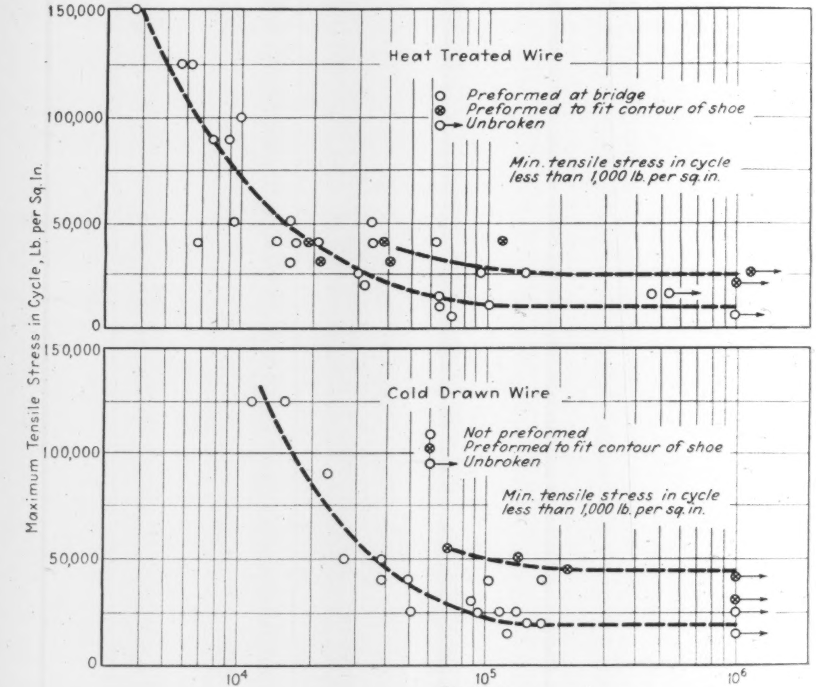


FIG. 7—FLUCTUATING TENSILE test results. Notable is the fact that only within the range 0 to 10,000 lb. per sq.in. was it possible to secure one million cycles on the heat-treated bridge wire without a fracture. Careful laboratory preforming raising the upper limit of the range to about 25,000 lb. per sq.in.

Poor preforming

In investigating these stress conditions 1,181 hairpins from the bridge were traced on a layout floor and the deviation between the curvature of the "strain free" or unloaded wire and its curvature when snugged close to the anchor shoe was determined. Not a single wire of all those examined was curved to fit accurately the curvature of the anchor shoe. Many had reverse curves; others contained rather sharp bends or kinks. In practically all of the wires, the point which had coincided with the midpoint of the circular curve on the anchor shoe was not at the midpoint of the curved portion of the wire. The preforming operation thus had failed completely in accomplishing its purpose in shaping the wire to fit the anchor shoe. In many cases the wire had been deformed so that when snugged close to the shoe, bending stresses were induced that were greater than if no attempt had been made to shape the wire. In contrast the cold-drawn wire hairpins were curved regularly and uniformly on a 40 in. radius when released from lateral constraint. Thus the bending deflections in many of the heat-treated wires, when they were forced from the strain-free shapes to the shape of the anchor shoes by a tensile pull on the wire, were greater than the bending deflections imparted to the cold drawn wires in the same manner.

Other results which pointed to the conclusion that the elastic bending stresses in the loops of the heat-treated wires were greater than those in the corresponding portions of the cold-drawn wires, were that permanent extensions in the cold-drawn wire were greater while for equal strain (in tension tests) stresses in the heat-treated wire were the larger.

However, the elastic bending stresses in combination with constant tensile loads or continuously increasing tensile loads were not sufficiently greater in the heat-treated wire to account for its failure on the same bridge where cold-drawn wire later proved adequate, so a further contributing cause was sought in fluctuating elastic bending stresses which would occur if the tensile load on the wires fluctuated from its maximum value, at any instant, to a lower value, provided that the decrease in tensile load permitted a change in the curvature of the wires at the shoes. Only circumstantial evidence is available to show that during the construction period the tensile pull fluctuated, although such fluctuations are known to be caused by variations in temperature, shifts in loading, and especially from wind forces occurring during the spinning operation. The maximum change in elastic bending stress would occur when there was a decrease in the tensile pull from a maximum value which was just sufficient to hold the wire closely to the contour of the anchor shoe, and such a condition undoubtedly occurred, in the opinion of the investigators, during the spinning of the cables. Tests to simulate this condition, therefore, were inaugurated.

Fluctuating tensile load tests

Fluctuating tensile loads were obtained in the laboratory by a crank arrangement which alternately removed and replaced suitable counterweights at the end of the lever arm of the testing apparatus shown in Fig. 6. The specimens were looped in the same way as for the long-time static loading tests.

This program of tests is summarized in Fig. 7. For the same conditions of curvature of the specimen and ranges of tensile load, no differences in results were observed between wire recovered from the bridge and the new wire. The galvanized coating was present on all specimens. A tensile stress of approximately 50,000 lb. per sq.in. was required to pull the specimens of both types of wire into position over the support so that a further increase in tensile stress caused no apparent change in the curvature of the specimen. In the range 0 to 50,000 lb. per sq.in., small changes in tensile stress caused visible changes in curvature of the specimens. The greatest change in curvature occurred when the tensile stress was changed from 50,000 lb. per sq.in. or higher to the minimum obtainable in the apparatus, 600 lb. per sq.in.

Comparison with cold-drawn wire

In general, at equal ranges of tensile stress, the cold-drawn wire endured a distinctly greater number of cycles before fracture than did the heat-treated wire. Although the results show a considerable scattering of the plotted points from a smooth curve, the stress-cycle curves show an increase in number of cycles as the maximum tensile stress of the range was decreased, with a definite trend to a constant range within which fractures were not obtained in one million cycles. For the heat-treated wire, this range was only from 0 to 10,000 lb. per sq.in., "a rather startling fact for wire with an ultimate tensile strength of 220,000 lb. per sq.in., and mounted as a tensile member in the same manner as it was installed on the bridge. This paradox is due to the fact that fractures were directly the result of the range of bending stress incurred when the tensile loads were alternately increased and decreased."

In all of the specimens of both types of wire, the fractures occurred without any deformation at the point of fracture. The surfaces of fracture had the distinctive characteristics of fatigue fractures with the point of origin at the surface of the wire. The appearance of the fractures in the heat-treated wire had a striking similarity to those that occurred on the bridge.

When the heat-treated wire specimens were preformed to fit the support, so that very little bending took place as the tensile loads were increased and decreased, the range of tensile stress to permit one million cycles without fracture was increased 2½ times, but even under these circumstances, the range, 0 to 25,000 lb. per sq.in., was less than the safe range of pulsating tensile stress under purely axial loading conditions. This indicates, it is concluded, that it was impracticable to preform this wire at the bridge so as to avoid damaging bending stresses in the loops when the tensile loads fluctuated.

For the cold-drawn wire, not preformed, the range of tensile stresses when the minimum was less than 1,000 lb. per sq.in. was also surprisingly low. When this wire was carefully preformed it was possible to raise the maximum tensile stress to 45,000 lb. per sq.in. without causing fractures in one million cycles. This range was almost as large as the safe range of pulsating tensile stress obtained on straight specimens that were tested in the Haigh machines.

As it was evident that the tensile stresses alone could not have caused the fractures, which were a direct result of bending stresses, observations were made of the behavior of specimens subjected to fluctuating tensile stresses but so constrained that the changes in curvature were as small as possible. By means of rigid braces fastened to both limbs of the U-shaped specimens (Fig. 6), practically no bending of the specimen occurred when the tensile loads fluctuated. Instead of breaking in from 500,000 to 1,000,000 cycles of tensile stress ranging from less than 1,000 to 10,000 lb. per sq.in., it was possible to raise the maximum stress of the cycle to 40,000 lb. per sq.in. without causing fractures in 500,000 cycles.

In summary "the results of the fluctuating tensile load tests showed that it was entirely possible to produce fractures in the bridge wire as the result of fluctuations in tensile load, when the tensile stress at no time exceeded the maximum calculated for the wire in the cables of the Mt. Hope bridge at the time it was dismantled. The fractures on the bridge were the result of the same *kind* of stresses as those that cause the fractures in the fluctuating tensile load tests in the laboratory."

Bridge Wire Tests Leave Some Questions Unanswered

Critique of Bureau of Standards Report

By Ernest E. Thum
Editor, Metal Progress
Cleveland, Ohio

NUMEROUS HYPOTHESES have been advanced as to the reason why heat-treated wire failed in suspension bridge service whereas cold-drawn wire has been uniformly successful. The principal ones were examined by the writer in four articles in *Metal Progress* starting in June, 1932.

It was shown that failure was not due to abuse during erection nor abuse during manufacture. It was not due to the use of steel unsuitable in analysis or melting practice. Furthermore, the constitution of the heat-treated steel was stable and did not change after the wire was accepted by mill inspectors. It is accepted that the surface smoothness of both varieties of wire is damaged by pickling and galvanizing, the heat-treated wire much more extensively than the cold-drawn. Finally, the failure seems to have resulted from conditions inherent in all parts of the wire as made—that is, to an essential difference between the microstructure of quenched and tempered versus cold-drawn steel. With these conclusions Messrs. Swanger and Wohlgemuth agree in their report of the Bureau of Standards investigation.

Slowly creeping loads

As a result of an intensive study made early in 1929, immediately after news of the failure, the writer concluded that the then discoverable facts fitted into the hypothesis that the fractures were due to the inability of heat-treated, galvanized wire to withstand a slowly creeping load. This hypothesis was discussed with numerous interested engineers and the investigators at the Bureau of Standards.

All straight wires curving around the anchorage shoes and some unfavorably placed ones squeezed under the cable bands at the suspenders were bent so that the extreme fiber was stressed beyond the elastic limit; this stress was never relieved—in fact, added to it was a slowly increasing tensile load as the bridge was completed. Similar conditions exist (although of somewhat less intensity, as Swanger and Wohlgemuth demonstrate) in cold-drawn bridge wire, but the essential difference, in the writer's opinion, is that the heat-treated wire is so stiff in its internal structure that it cannot yield readily at minor surface imperfections (acting as "stress-raisers,"), and redistribute the stress to neighboring regions in the cross section. Hence a splitting type of fracture with no contraction of area ocurs under slowly creeping loads beyond the elastic limit. Cold-drawn wire, on the ohter hand, is much more plastic and has a low creep limit at atmospheric temperature, so the danger from scratches, notches or slag specks at the surface is much less.

If this hypothesis were true, it should be possible to duplicate the fractures occurring on the bridge by laboratory tests. H. F. Moore tried it at the University of Illinois on a few samples of spring wire —none of the suspicious bridge wire being procurable by him—with negative results, (*Metal Progress*, Sept. 1933). On the other hand W. A. Slater and M. O. Fuller at Lehigh University tested nine pieces of the heat-treated bridge wire bent around a 9¾ in. radius and then loaded by increments starting at 84,000 lb. per sq.in. One broke while hanging overnight under the initial load, none withstood more than 201,500 lb. per sq.in., and all broke with a silky fracture, angular (no cup and cone), and no reduction of area.

Five testing machines were built at the Bureau of Standards for testing loops of wire, and 135 tests were made, with step-up loads starting at 90,000 lb. per sq.in. Only eight broke at less than the nominal ultimate tensile strength and none at less than 120,000 lb. per sq.in. (after an increment of 30,000 lb. per sq.in.), while 34 had less than 20 per cent reduction of area and therefore were classed by the investigators as a "brittle type of fracture." From their own tests Messrs. Swanger and Wohlgemuth concluded that "the results of the static [and step-up] loading under combined [flexural and tensile] stresses cannot be accepted as an explanation of the mechanism of failure of the wire on the bridge."

The present writer would have proposed the word "complete" before "explanation" in the above quotation. As will be developed later, and even in the light of the tests at Lehigh and Washington, creeping loads cannot be entirely absolved from guilt. It is noteworthy that the difference between ultimate strength of heat-treated wire and its yield point is 223,000 —189,800 or 33,200 lb. per sq.in. The lowest breaks experienced at the Bureau of Standards occurred after stepping up the load 30,000 lb. per sq.in. Lastly, the computed load on the cables at Mt. Hope when fractures were first found was 32,000 lb. per sq.in. This agreement of figures may be accidental, but then again it may have some significance as to the minimum tensile strength of this heat-treated wire when a tensile load is superposed on bending stresses at and beyond the elastic limit.

Fluctuating tensile loads

By further consideration of the special conditions existing where the bridge wire starts bending around the anchor shoe (the locus of most of the trouble), Swanger and Wohlgemuth were led to the supposition that the tension was not relatively quiescent but rapidly fluctuating. Extensive and valuable fatigue tests on both heat-treated and cold-drawn wire were then made and developed their endurance as galvanized, as stripped, and as machined down to solid metal. It was discovered that a hair pin of heat-treated wire, bent around an anchor shoe and pulled taut, would break if the pull were fluctuated between 600 and 10,000 lb. per sq.in., whenever the experimental set-up was such that this range in tensile stress permitted the bent wire at the shoe to change its curvature by "spring back." In other words, minor changes in tensile stress induced major changes in flexural stress, beyond the endurance limit of the material, and fatigue failure resulted.

Some further questions

Since the investigators were able to break the heat-treated wire in laboratory tests at low tensile loads, and the fractures certainly looked like the ones that occurred on the bridge, the presumption is very strong that they have hit upon the right track. The only questions left to consider are:

(a) Did rapid fluctuations of 10,000 lb. per sq.in. actually occur in the bridge wire?

(b) Why did so many wires break under the cable clamps at the suspender wires?

(c) Why were the breaks at the anchorage concentrated largely in strand No. 1 of the East cable at the Bristol end of the bridge, and in strand No. 3 at the Portsmouth end of the West cable?

Magnitude of fluctuations

In answer to (a), is is unfortunate that Messrs. Swanger and Wohlgemuth could not extend their paper to include a mathematical analysis of the vibrations in the Mt. Hope bridge wire caused by side winds, the prinicpal cause of rapid fluctuations, and an estimate of their effect on the tensile load. Lacking such analysis it is hard to imagine conditions that would cause a relief of 10,000 lb. per sq.in. in direct tension, when the stress in the wire due to its own weight is no more than 5,360 lb. per sq.in. at the anchorage. Wind vibration effects would probably be a maximum during early stages of spinning, when the individual wires are hanging free, for as soon as a strand is spun it is bundled together at intervals along its length, and of course the complete cable is tightly compacted and bound into a quite stiff unit. Yet no breaks occurred in the cable in this early, flexible stage; they were first observed when the bridge was nearing completion and the whole thing was relatively rigid.

As to (b), it is believed that the Bureau's investigators have paid too little attention to the breaks found along the cable, away from the anchorage. They say of the Mt. Hope bridge: "Approximately 20 were found under the saddle clamps at the cable bents. None were found at the main tower saddles. Eleven were found under cable bands in the side and main spans. Most of these breaks were at the outside of the cables and were believed to have occurred at points of localized stress where the saddle covers or bands had pinched the wire . . . and probably were not very significant." On the Ambassador bridge a considerably larger number of broken wires were found at the suspender points.

The creeping stress theory explained these breaks in rather sharply curved wires near the surface, where the cable is squeezed tighter by the bands and the slowly rising stresses of subsequent bridge erection. If fluctuating tension in bent wires is responsible, the action cannot commence until the wires actually are bent (that is, until the clamps are tightened up), and consequently the damage started much later than at the anchorages and fatigue failure occurred in a comparatively short time in stiff, compacted cable—all of which seems rather unlikely.

On the other hand severe lateral compression and a fluctuating load superimposed (either in tension, bending or both) may be an irresistible combination and cause quick fatigue failure. Some hitherto unexplained breaks in axles inside the bore of wheels which were forced on with a pressed fit may be instances. Also serious troubles with cold-drawn stay wire bracing in aircraft and dirigible skeletons involve fatigue failures with nucleus frequently on the inner curve of an end loop, where it bears against the anchorage pin. In the latter case the cure is to hammer the loop on the outside so it is snuggled close to the grommet or to fit and securely solder the loop around a stiff ferrule of much larger radius, so the bearing is ferrule against pin, and wire against ferrule, and the whole assembly much stiffer against bending under vibrating tensile loads. (Even better is to substitute an upset and threaded end fitting.)

It should also be remarked that the wires around the anchor shoes in a suspension bridge (except the outside row) are nested together in such a way that they are tightly constrained laterally.

Many breaks in one strand

The last question concerns the concentration of so many breaks in one end of one strand of a cable. It is yet to be explained, although it may readily be shown that the unit load increases on the wires in a strand at an anchorage where there are broken ones (and increases rapidly as more wires break) while the stress conditions at the undamaged end of the cable are not changed, due to the friction between wires along the compacted cable. The stress at the damaged anchorage is further increased by the elastic contraction of the eyebars attached to the strand shoe in question. Computations of stress conditions in strand No. 1 show that the first fractures (20 loops) increased the load on the remaining wires in the strand 7.2 per cent; that breakage of 65 more wires raised this to 26 per cent over the 32,000 lb. per sq.in. initial stress; and that when three-quarters of the wires in this strand were broken, next day, the unbroken wires carried an 84 per cent stress increase. The concentration of many breaks in one strand may consequently be explained on the grounds that this strand accidentally developed a few breaks early in the history of the failure and thus was overloaded progressively faster than the other strands. The concentration of breaks at one end of a strand may be explained by the friction of wires in the compacted cable.

More study needed

This rather brief review of some pertinent facts in this historic bridge failure indicates that, despite the enormous amount of careful and well-planned work done at the Bureau of Standards, a completely satisfactory explanation of the difficulty is not yet forthcoming. As before mentioned, the ability to develop fractures in the laboratory that look like the broken wire-ends from the bridge anchorages and at a combination of high bending and low tension loads indicates that we are close to the correct solution. It does not appear to be, however, the complete solution. In a number of instances, other factors also had major influence. What these factors are is still hypothetical. One may be heavy lateral constraint against movement at the surface. Another probably is a slowly creeping tensile stress.

Vulnerable though the bent heat-treated wire has proven to be to fluctuating load with unchanged average intensity, it may be even more vulnerable when the average intensity is gradually rising. A series of "step-up fatigue tests" would determine this point.

Surface imperfections

Lastly, the surface condition of the heat-treated wire was shocking. This refers not to the minor pits and scratches but to the deeper cracks which infested the wire and caused one out of every seven to ten tensile specimens to break with practically no reduction of area. Even though Messrs. Swanger and Wohlgemuth cannot find that these deep cracks affect the endurance of the wire, they were undoubtedly responsible for a large proportion of the rejects at the mill, and this would have been more ruinous if retests had not been allowed. (17% of this wire was rejected, as compared with 1.5% rejects of cold-drawn wire shipped from the same mill to the Delaware River Bridge!)

Spring makers find similar cracks in the surface of heat-treated springs after electroplating—possibly it is a characteristic action of the cleaning acid on the surface of any heat-treated wire, no matter how well made. However the prevalence of such cracks is probably sufficient to place one of these major surface defects within the region of stress concentration for practically all of the breaks that actually occurred on the bridge, and possibly was the immediate cause of the entire trouble.

Now what metallurgists and designers would like to know, even though bridge engineers will wisely stick to cold-drawn wire, is whether heat-treated wire with really good surface, either bare or rust-proofed without surface damage, would be immune to the troubles that plagued the Mt. Hope bridge? In other words, were the failures due to the essential nature of the microstructure (relatively stiff and unyielding) or is this characteristic harmless unless the metal has gross surface imperfections?

Even though some questions remain unanswered, the researches so far have clear lessons that point to certain desirable changes in the design of suspension bridges. The relatively small margin of safety of cold-drawn wire (accepting all the implications of Fig. 41) indicates that both bending and fluctuating stresses should be minimized as much as possible. This will involve three things:

(1) Anchor shoes should have as large a radius as possible.
(2) Stiff "keepers," carefully machined to correct line and curvature, should enclose all strands from within the splay casting to and around each anchor shoe.
(3) Eye bars should be concreted into the foundation as far up toward the anchor shoe as possible.

Lastly, the practice of allowing a retest may be closely scrutinized. A plausible argument might be made for one when the original test fails to meet the specified values by a small margin once in 100 times. But nothing can be said of a practice which permits the acceptance of material which shows deficient ductility in 10 per cent of the tensile tests.

Cisterns Store Water for Fires Where Earthquakes Menace Mains

San Francisco has built a system of underground storages distributed throughout the city—Cisterns located at street intersections hold 75,000 gal. and cost $6,000—Pipe connections omitted for safety

WORK NOW is nearing completion in San Francisco on a cross-town line of cisterns intended to increase protection against a general conflagration that might occur after an earthquake had broken the mains and interrupted the water supply as was the case in 1906. This present extension of the auxiliary water supply system is part of an improvement plan for which a $2,000,000 bond issue was voted in 1933. Nineteen additional concrete cisterns are being provided and these, together with some previously constructed, will afford a chain of underground water storages not more than 1,000 ft. apart extending entirely across the city in a north and south line. This line runs southward from the Bay along the full length of Van Ness Ave. to Market St., along Market to Dolores St. and thence southward on Dolores St.

Cisterns are a heritage from the early days in San Francisco. Before there was an adequate water distribution system, underground storages were important items in the fire protection plan. Six such cisterns are shown on the official map of the city in 1852, largely in residential districts. Between that time and up to about the break of the century 56 more cisterns were added. These were of brick construction and had capacities varying from 14,700 to 106,000 gal.

Unfortunately, in the years just prior to 1906 attention to cisterns lapsed because the domestic water supply distribution mains and the hydrant connections were improved and enlarged to such an extent that fire protection seemed assured. Cisterns then fell into disuse. By 1906 some of them had been filled with débris and the locations of others were unknown. Thus the great conflagration that followed the earthquake of April 18, 1906, found the city unable to cope with a situation in which the water supply failed as the result of ruptured water mains. If an auxiliary supply in cisterns had been available it is believed that this would have made it possible to hold the fire in check and perhaps to have prevented the general conflagration that swept the city.

In 1908 a bond issue of $5,200,000 was voted to construct an auxiliary water system for fire protection entirely independent of the domestic water supply. A major feature is a system of high-pressure mains with pumps and connections designed for taking an emergency salt water supply from San Francisco

Bay. The old cisterns were repaired and reconditioned for service; today most of them are available for use. Included in the program were 86 new cisterns at strategic locations throughout the city. These were built according to a standard design in reinforced concrete at a cost of about $6,000 each. The standard capacity is 75,000 gal. and the intent is that two pumping engines can draw simultaneously from each cistern. No fixed pipe or other connections were included in the design, thus avoiding risk of disruptive stresses which might occur as the result of earth movement. The reinforced concrete structure can move as a whole without injury. This was proved during the construction period when water from a rain storm floated one of the cisterns 6 ft. off its foundation. After it had settled back into place it was still intact and had developed no leaks.

The standard design calls for a cistern with an inside diameter of 30 ft., a maximum depth of 18 ft. and top and bottom in the form of opposed domes. Two manholes are provided, one at the top center and one at the outer diameter. Usually the cisterns are put at street intersections and the location is clearly marked by a ring of basalt blocks 30 ft. in diameter set in the pavement.

In 1933 another bond issue of $2,000,000 was voted for further extensions to the auxiliary water supply system. Included in this sum was an allotment for 19 additional concrete cisterns to be built in accordance with the standard design mentioned. In planning these later structures locations that had strategic advantage from the distribution standpoint were selected, ground conditions were studied and modifications of the design were made where required. The location of existing subsurface installations was considered to avoid unnecessary changes of utilities already in place.

When the present program is completed the concrete cisterns of standard design constructed since 1906 will have a combined capacity of 8,125,000 gal. If the old brick cisterns built previous to 1906 are included the total capacity is 10,025,000 gal. In addition to this and exclusive of storage in tanks and reservoirs of the domestic water supply system, there are storages of fresh and salt water in parks, swimming pools, etc., totaling 52,000,000 gal.

Providence Report Favors Extension of Water District

A REPORT recommending that the Providence Water District be extended to include the towns of North Providence and Johnston together with the city of Warwick has been made following a study looking toward the extension of the distribution system beyond present limits to secure greater daily consumption. The actual daily consumption of only 27 m.g.d. from the Scituate reservoir district in 1935 against the original estimate of the water supply board of 37 m.g.d. for the same year, seriously handicaped the finances of the division and prevented the proper income which should have been derived from the system in accordance with the plans when the original investment was made. Before extending the district, however, it was essential to determine whether or not the diversion of additional water would jeopardize the city's contractual agreement with mills below Scituate dam.

In the report made by Philip J. Holton, Jr., under the direction of Charles A. Maguire, commissioner of public works, it was concluded that the city might extend service to other districts outside the area specified in the act of 1915, without jeopardizing the interests of the mill owners or the city. Diversion claims have been paid, based upon complete diversion by 1955 with a consumption of 61 m.g.d. by this date. Comparative studies show that by serving consumers in the additional areas, the consumption in 1960 would be only 35.3 m.g.d. Even on this date, with the additional water that will be diverted for serving the increased population, the mills will be receiving water far in excess of their contract requirements and for which the city has paid over $2,000,000 in claims.

It was stated that the possible growth of Providence is questionable as the city is somewhat saturated, with no large areas for residential or commercial development that would impose a burden on the water system. During 1935 the daily consumption averaged only 27 m.gd. against a daily reservoir yield of 111 m.g.d., consequently there is lttle possibility of the city being compelled to obtain additional water supplies.

If the extension to the water district is approved, the increase in consumption would be approximately 2 m.g.d. by 1940. This increased daily consumption would mean an additional income to the water department of $175,000 annually. At present some income is derived from excess water passing the dam through the development of power at the gate house. There is no comparison, it was pointed out, as to the income that may be derived through the different methods. One million gallons of water used for power generation represents an income of only 80 cents, whereas one m.g.d. for domestic consumption represents an income of $240.

Educational Qualifications of Engineers

IN MAY, 1935, the Bureau of Labor Statistics undertook a survey of the engineering profession at the request of the American Engineering Council. It was desired to determine the extent of unemployment, what kind of professional employment gave engineers the greatest protection against unemployment, and their compensation between Dec. 31, 1929 and Dec. 31, 1934. An investigation also was made of the education and subsequent experience of engineers. A compilation of the results covering education and functional distribution of engineers was published in the June, 1936, issue of *Monthly Labor Review*. The data on which the study is based were obtained through a mail questionnaire addressed to engineers throughout the country.

The data make clear the increasing importance of an engineering education. Of 19,891 civil engineers reporting, only 5,191 were not graduates of an engineering school. Moreover, a breakdown as to age groups shows a much smaller percentage of non-graduates among the younger than among the older engineers. Thus, while there are 113 non-graduates among the groups born between 1910 and 1914, there are 691 non-graduates among the groups born between 1890 and 1894.

The report also brings out that only a very small number, 8.4 per cent of those reporting, shift from the field in which they obtain their education to another engineering field. Also interesting is the very small amount of graduate study revealed by the survey. Of civil engineers possessing college degrees, 2.9 per cent possess an engineering degree plus a B.A.; 0.03 pr cent an engineering degree plus M.A.; and 0.1 per cent an engineering degree plus Ph.D. Only 1.5 per cent possess an M.A. in engineering, and 0.2 per cent a doctor's degree in engineering.

The classification of engineers by field of activity reveals that 63.3 per cent of civil engineers are engaged in government work; the next highest classification, construction, is 15.4 per cent. The functional classification shows 45.7 per cent of civil engineers in construction work, 21.8 per cent engaged in design and research, and 12.5 per cent in operation.

MOSQUITO CONTROL ENGINEERING . . . III

Control Technique and Organization

The nature and methods of practical mosquito control are a highly specialized type of operation predicated upon the application of scientific knowledge coupled with broad experience.

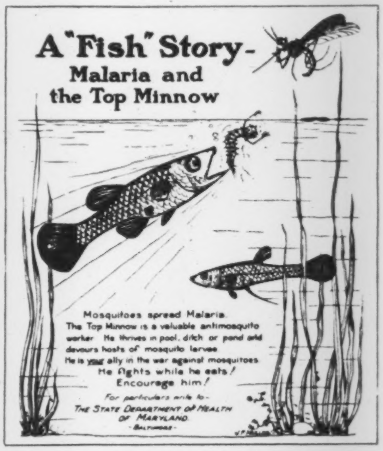
By Russell W. Gies
Sanitary Engineer,
Department of Health, New York City

IN THE PREVIOUS ARTICLES, Dr. L. O. Howard, for many years chief of the U. S. Bureau of Entomology, traced the development of mosquito control work from its beginning less than forty years ago to its present status as a world-wide crusade, and Dr. T. J. Headlee, state entomologist of New Jersey, discussed the biological factors underlying our present knowledge of control technique. It is the purpose of this article to outline the nature and methods of practical control which have been formulated through the application of scientific knowledge coupled with broad experience. As in any other type of specialized operation, it is proper organization and effective administration that determine in large measure the success of a mosquito abatement program.

Mosquitoes need water

All mosquito control work is correlated with the fact that mosquitoes breed in water and in water only. Consequently all water that harbors mosquitoes should either be eliminated or so treated that it will be incapable of supporting further breeding. It is in the breeding stage that this insect is most vulnerable and therefore can be eradicated at the least cost.

Mosquitoes do not breed in every kind of water. For instance, they will not propagate in dew-moistened grass areas as is popularly believed—the water must be at least ½ in. deep. They have not been found to breed in clay pits or cedar swamp water (unless polluted) probably due to certain toxic substances or the lack of suitable food. Neither will they breed in strong sewage until it has been somewhat diluted. Fresh water varieties are seldom found in water of greater salinity than about 6 per cent and salt marsh mosquitoes do not develop in fresh water unless it is polluted. Water temperature is important and must reach 40 deg. as a minimum (varying slightly with the species) before the eggs will hatch. The water must also be available for at least seven days and remain stagnant or slow moving to permit breeding. Mosquitoes do not customarily breed in water which is rippled by wind, probably because this action interferes with the laying of eggs or with proper breathing. Finally, mosquitoes will not develop in places where fish are present and able to reach the sheltered areas that harbor larvæ.


A "Fish" Story-
Malaria and
the Top Minnow
Mosquitoes spread Malaria.
The Top Minnow is a valuable antimosquito worker. He thrives in pool, ditch or pond and devours hosts of mosquito larvae.
He is your ally in the war against mosquitoes.
He fights while he eats!
Encourage him!
For particulars write to-
THE STATE DEPARTMENT OF HEALTH OF MARYLAND
-BALTIMORE-

EDUCATIONAL PROPAGANDA is an effective aid in winning community support for anti-mosquito work. The above is from a poster used by the Maryland State Health Department.

Breeding places, in general, can be classified in two groups—permanent and temporary. Under the former we have collections of water such as ponds, swamps, stagnant brooks and rivers, roadside ditches, open tanks, sewer catch basins, cesspools, cisterns, etc. The so-called temporary places include a wide variety of water-holding receptacles such as barrels, tubs, discarded rubber tires, bottles, and roof gutters—the qualifying factor is that water remains in such places for at least seven days during warm weather. The number and importance of temporary places as a source of breeding is usually underestimated, yet these temporary places account for at least half the number of mosquitoes present in most communities.

The prerequisite information to answer the question of how to rid a locality of mosquitoes is definite knowledge of the kinds of mosquitoes present in the area to be protected. Breeding habits of the various species differ so widely that without this basic data an eradication campaign may be entirely unsuccessful despite the expenditure of considerable sums of money. To secure this information a survey of the area is made which involves the collection of mosquitoes on the wing to determine their species and preparation of maps showing the location of all breeding places.

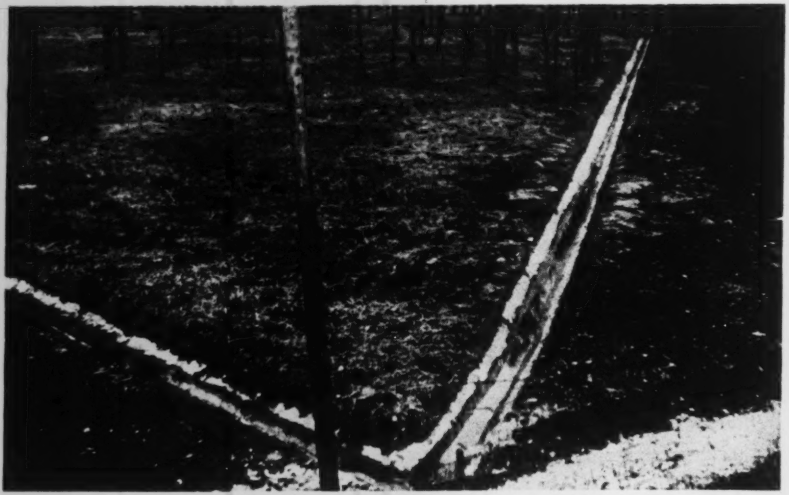
Other considerations preliminary to the inauguration of an abatement program are: the distribution of population; the amount of annoyance or illness caused by mosquitoes; the most practical size of the area to be controlled; the cost of the proposed permanent and temporary work, and the manner of financing the undertaking.

The technique of trapping adult mosquitoes has been thoroughly described by Dr. Headlee, in a previous article (*ENR*, Aug. 6, 1936, p. 199). At the time when these trapping operations and collections are being made it is also advisable to prepare a map of the places, especially the larger ones, where definite mosquito breeding has been observed. This involves thorough scouting over the areas to be protected and possibly the inspection of adjacent region to determine if migratory species are in evidence. A U. S. Geological Survey map has been found to be of considerable aid in making preliminary surveys and considerable information can be secured from these maps as to drainage possibilities. A larger scale map, however, is usually better suited for the purpose of recording the data secured during the course of the survey. While this work is going on information should be compiled regarding the cost of permanent abatement measures through drainage or filling.

Zone of operations

The size of the area to be placed under control will depend in large measure on the evaluation of the factors mentioned above. It has been found, in general, that the larger the area to be protected the lower will be the unit cost. While the county unit, the abatement district (comprising several cities or townships) or a state-wide organization has been found to be the most satisfactory, there is no reason why a single small village, under intelligent direction, can not pursue an effective abatement program at reasonable cost. The actual cost of the control program will depend on: the

PRETTY AS A PICTURE but the birthplace of a summer plague. Stagnant woodland pools and water-courses in city parks encourage prolific mosquito breeding. In the lower view is shown a type of concrete-lined ditch that has been found to be an inexpensive means of draining park grounds to permanently eliminate breeding possibilities.



density of population, the number of natural breeding places, the required amount of inspection and temporary eradication measures, and the necessity for carrying out educational work to secure public cooperation.

Mosquito control work varies in cost from 5c. to $1.00 per capita yearly, with an average of about 25c. per capita. Whatever money is obtained is usually budgeted into two parts; namely, a fund to take care of the temporary abatement measure during the breeding season, and a sum to be used for permanent work in connection with drainage or filling. Temporary work such as inspection, treatment with larvicide and maintenance of existing drainage should be immediately started in order to reduce the number of mosquitoes to a point where improvement is noticeable. If this is not done, the faith and support of the people in the protected area is apt to be lost and further appropriations will be difficult to obtain. As much money as possible should be spent on permanent drainage or filling in order to reduce the necessity for temporary work in the future and thus ultimately reduce the cost of the whole mosquito control campaign to a minimum amount each year. It should be thoroughly understood that a mosquito control campaign must be continued and maintained from year to year —it is a never ending warfare against a very persistent enemy.

Selling the idea

Assuming that we have reached the point where all the basic information has been obtained, the next step is to sell the idea of starting work so that the necessary funds will be found. Usually, the best procedure is to secure a small group of prominent local people who are interested in getting rid of mosquitoes and who will sponsor the campaign. If such a group can be secured—a group that commands the confidence of the community—it can enlist the support of the newspapers, women's clubs, chambers of commerce, departments of health, hotel and real estate men. One of the most effective selling arguments for securing the necessary funds has been found to be the recital of economic benefits. Mosquito control work pays in dollars and cents, not only in reducing the amount of malaria and other diseases which may be carried by mosquitoes, but also in increasing real estate values. What community is not more attractive without mosquitoes?

Some years ago the New Jersey state department of conservation and development made an investigation from which it was estimated that through mosquito control work there would be an increase in land values, benefitting agriculture, seashore industries, industrial and urban development, amounting to $507,000,000. The cost of the necessary mosquito control work was computed at a very small fraction of this increase in ratables.

Financial aid for carrying on mosquito abatement may be secured either through city, town or village budgetary funds, voluntary contributions, state appropriations, or federal allotments. The WPA has looked with considerable favor on mosquito control work as one of the desirable types of relief projects because the money required for materials and equipment has been very low in proportion to the amount spent for wages. In some states, CCC camps have been provided by the federal government to carry on drainage work in connection with mosquito control projects.

Organizing for effective work

After the necessary financing has been taken care of the next step is to build up an effective organization. In many states mosquito abatement districts have been organized with public spirited men and women in a non-paid capacity acting as commissioners; the actual conduct of the work is usually left to a thoroughly experienced technical director, whose background combines both engineering and entomological training. In a number of cases, excellent mosquito control programs have been carried on and supervised by local health forces and municipal engineers.

The personnel of the organization, in addition to the technically trained and paid director, will vary with the size of the abatement district and problems involved. The fundamental principles, however, are the same for a small village or a town as for a large city, district or state. First, there must be a definite amount of inspection to locate the breeding places and to provide weekly visits to these spots during the breeding season; second, there should be a regular larvicide squad for temporary work to see that the breeding is actually under control at all times; and third, there is needed an engineering or drainage service to effect the permanent elimination of as many breeding spots as possible. In addition, when covering a large area, it is advisable to institute an entomological survey service which will be principally engaged in making collections of adult mosquitoes in order to determine in what areas the most effective work is needed. A publicity or educational bureau should also be maintained in order to keep the citizens of protected areas acquainted with what is going on and thus secure their active cooperation in eliminating breeding places around the home. These are the fundamental elements of organization which experience has shown to be necessary—a mosquito control machine developed efficiently along these lines should get good results.

Temporary control work includes thorough periodic inspections to locate all breeding places and the application of larvicides. The region to be controlled should be divided into districts of such size that an inspector can cover the entire area at weekly intervals. Usually an inspector covers from 1 to 10 square miles depending on the distribution of breeding places, the type of terrain and weather conditions. Not only should the larger breeding areas be visited once a week but where a sufficient number of men are available it is customary to inspect as many back yards as possible; even in the high class residential neighborhoods a surprisingly large number of breeding spots will be found.

The larvicides in general use consist of either light fuel oils or solutions containing an oil extract of pyrethrum. The latter, developed largely through the efforts of Dr. J. M. Ginsburg, biochemist at the New Jersey Agricultural Experiment Station, has to a considerable degree superseded the fuel oil formerly used. It is cheaper than oil in most localities, it does not affect vegetation and it is non-injurious to water fowl and fish life. This highly toxic larvicide in dilute solution is sprayed over each breeding place once a week by means of hand or power spray equipment.

Very recently the same type larvicide has been successfully used for protecting outdoor public gatherings from adult mosquitoes by applying it in a fine spray to the atmosphere, trees, bushes, and ground. One application is often sufficient for an entire evening. If necessary the lavicide can be applied several times during the course of an evening without causing any harm to clothing or people.

Natural enemies of mosquitoes in their early stages of development are various kinds of fish. On the flooded expanse of salt marshes along the Atlantic, Gulf and Pacific coasts, will be found the salt water killy fish (*Fundulus*) or the sheepshead minnow (*Cyprinodon*); they are present in enormous quantities and readily devour all mosquito larvae. The fresh water variety of *Fundulus*, "shiners" as they are commonly known, are distributed over the north portion of the United States and they are highly useful as a control agent. In the South, the top feeding minnow known as *Gambusia* is generally recognized as most effective. Permanent collections of water stocked with any of these varieties of fish will not breed mosquitoes, providing the shoreline is kept free of vegetation so that the fish can penetrate into every nook. In many parts of the country the rainwater barrel fish (*Lucania*) is found, and one or two of these in a rain barrel will keep it free of mosquito breeding.

Permanent work directed toward the control of mosquito breeding should proceed concurrently with the temporary measures and should be carried on as rapidly as possible. This type of work involves drainage and ditching both on the salt marshes and uplands, the construction of dikes and tide gates, and filling. This matter will be discussed in subsequent articles.

The New York City campaign

Probably the largest organized anti-mosquito campaign going on at the present time is the work which is being done in New York City. Within the limits of this great metropolis there are many conditions conducive to mosquito production. About 22,000 acres of salt marshes, nearly twice the area of Manhattan Island, are located within the city limits, and until recently, there existed about 8,000 large fresh water breeding spots exclusive of some 60,000 sewer catch basins. Over a period of years since 1905 the city has been slowly ditching the salt marshes and filling some of the fresh water areas but with the advent of the depression and its accompanying relief problem, this work was undertaken on a large scale basis. Since 1933 several thousand men supplied by CWA, FERA and now the WPA, have been steadily employed in the warfare

MOBILE SPRAYING unit for mosquito control recently adopted by New York City. Concentrated larvicide is added to the 550-gal. tank which is then filled with water. This solution is pumped at the rate of 30 gals. per min. through 500 ft. of hose at a pressure of 350 lb. per sq.in.

"BEFORE AND AFTER" the mosquito inspector made his report. Every municipality has dozens of eyesores, such as shown on the left, where countless hordes of mosquitoes will breed in a season.

against mosquitoes; as many as 7,500 men have been used at one time.

The relationship between the New York City health department, which acts as sponsor for the city in connection with this work, and the WPA has been in the nature of owner and contractor. The health department through a division of mosquito control in the sanitary bureau has been responsible for the design, planning, the general scheme of organization, the inspection of the work as it progresses and final acceptance. The WPA has been responsible for the actual carrying out of the work which includes the location of breeding places and application of larvicides until permanent drainage or filling has been completed, and the mapping of all salt marsh and fresh water drains and ditches. In addition, an extensive entomological survey is being conducted and general educational work has been promoted.

The same type of organization and the principles of control that are being utilized to protect the 7,500,000 people of New York City can be adopted on a smaller scale by any community that is sufficiently interested to free itself from mosquitoes.

The fourth article in the series, written by J. Lyell Clarke, sanitary engineer of the Des Plaines Valley (Ill.) Mosquito Abatement District, is to appear next week and will describe the manner of controlling inland species.

Pyrethrum Extract Emulsion For Anti-Mosquito Spraying

Method of preparation and the use of this material as a larvicide on breeding areas and as an insecticide against adult mosquitoes

IN THE series of articles on mosquito control of which the preceding article is a part, reference is made to the use of pyrethrum larvicide as a means of killing mosquito larvæ and pupæ in water where breeding is taking place. Quite recently this chemical has also been adopted for use as an insecticide in protecting outdoor gatherings from annoyance by adult mosquitoes. It is of general interest, therefore to describe the preparation and use of the oil pyrethrum extract emulsion as developed by Dr. J. M. Ginsburg, of the New Jersey Agricultural Experiment Station, New Brunswick, N. J.

The stock emulsion consists of the following ingredients mixed in the proportions given:

Kerosene	90 gal.
Kerosene pyrethrum extract (analyzing 2 per cent pyrethrins)	5 gal.
Sodium laurel sulphate	6 lb.
Water	50 gal.

Larvicidal use

The emulsifier (sodium laurel sulfate) is first mixed with the water and transferred to a tank equipped with agitators or a pump (preferably with both), which takes solution from the bottom of the tank and returns it to the top. The oil is then emptied slowly into the tank with agitators and pump working at full speed. After all the oil has been added the pumping is continued until the entire mixture is thick and homogeneous, showing no free oil on the surface. Excessive foaming may be eliminated by dissolving about 2 or 3 pounds of wool grease in the kerosene before emulsifying. The finished product, pumped into drums for storing, constitutes the stock emulsion.

If the larvicide is to be used only on fresh water, or on waters of very low salinity, soap can also be used as an emulsifier. In this case it will require about 1 per cent actual anhydrous soap for the emulsion, following the same procedure as outlined above.

The stock emulsion may be diluted with water and sprayed on various breeding places such as sewage disposal plants, standing pools, ponds, edges of rivers, streams, etc. It has been found that a dilution of 1:10 or lower, with thorough application, gives complete kill of larvæ and pupæ. At this dilution, it is harmless to fish, water fowl, and plant life.

Protecting outdoor gatherings

As an insecticide against adult mosquitoes the following procedure has been found effective: About half an hour before the public gathering takes place the area is completely sprayed with the larvicide diluted 1:10 or 1:12, that is, 1 part of stock emulsion is mixed with 10 parts or 12 parts of water. The spraying is done with a power sprayer capable of developing a pressure of 100 lb. per sq.in. and equipped with a spray gun and fine nozzles. Before mixing with water, the concentrated stock emulsion should be well shaken; the diluted larvicide should also be frequently stirred or agitated in order to secure uniform distribution throughout the spraying operation. The spray is applied in the form of a fine mist directly to the grass, grounds, tents, trees, shrubs, etc. Then the stream is directed upward so as to saturate the atmosphere with the mist. At no time should a coarse spray be applied, since it is unnecessary and may injure vegetation.

The grounds for a distance of about 20 ft. outside the area should also be thoroughly fogged, especially when tall grass, shrubs, and other vegetation are present which offer a hiding place from which adult female mosquitoes may issue suddenly at dusk in large numbers. If the area has been thoroughly fogged one treatment may suffice for two hours or possibly the entire evening. If mosquitoes become bothersome later in the evening, the area on the outside of the "gathering" grounds should again be fogged, directing the stream primarily upward and towards the ground to be protected. This outside fogging may be repeated again if necessary.

On small areas, such as back yards, private lawns, etc., a knapsack sprayer or bucket pump capable of producing a fog spray 10 to 15 ft. in height can be used. Before attempting to spray, a survey of the area should first be made in order to ascertain the main source or sources where mosquitoes are apt to originate. After the entire area is covered, the spraying outfit is kept in readiness near these anticipated sources in order to ward off a sudden onrush of mosquitoes.

Nonflammable Pyrethrum Spray For Use in Airplanes

NEW PROBLEMS in controlling the spread of quarantinable diseases brought about through the increase of international travel by airplanes are being gradually solved by the U. S. Public Health Service. In the *Public Health Reports,* Vol. 50, No. 41, p. 1401, C. L. Williams and W. C. Dreessen describe studies made in connection with the destruction of *Aëdes aegypti* mosquitoes which transmit yellow fever.

In order to destroy these insects on airplanes while in flight it is necessary to have an agent which will kill the mosquitoes without hazard to human occupants. Pyrethrum in a kerosene base answers this purpose in certain respects in that it is innocuous to human beings while at the same time lethal to *Aëdes* in low concentrations. However, it has the serious drawback of being flammable.

Experiments were recently made using a spray of pyrethrum extract (2 per cent pyrethrins) in a carbon tetrachloride base which is nonflammable. As a result of these tests it was stated that the lethal concentration of this agent for *Aëdes aegypti* was found to be between 40 and 60 cc. per 1,000 cu.ft. The latter amount was too great however, to be tolerated by observers who were accustomed to remain in the room during the spraying operation.

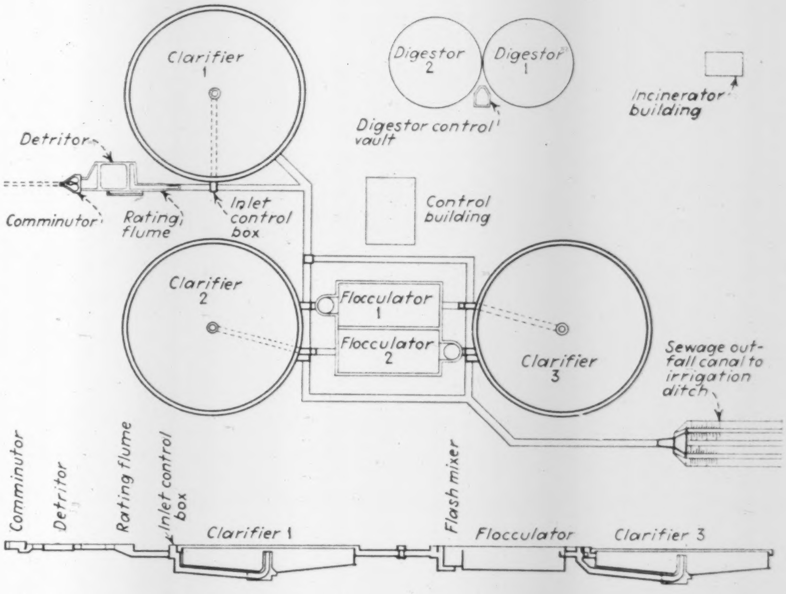
With kerosene as a base (2 grams of pyrethrins per 100 cc.) it was found that only 2 to 4 grams per 1,000 cu.ft. were lethal to *Aëdes* when used as a fine spray.

Low Cost Chemical Precipitation

Colorado Springs, Col., with a heavy seasonal variation in flow, adopts chemical precipitation using two-stage flocculation and three-stage sedimentation with combined sludge and garbage incineration

By Burton Lowther
Consulting Hydraulic and Sanitary Engineer, Denver, Colo.

CHEMICAL precipitation of sewage, at a unit cost comparable to most primary treatment processes, influenced Colorado Springs, Colo., in adopting the unusual design for the plant now under construction. Features such as two-stage flash mixing and flocculation, three-stage sedimentation, flocculation to improve plain subsidence, multiple digestion, filtration and incineration of sludge with garbage contribute to the distinction of the process selected.



FLEXIBILITY of operation methods to meet greatly varying volumes of sewage flow, characterizes the layout of the Colorado Springs chemical precipitation plant.

Into the Fountain River, the principal stream of the immediate area, and tributary to the Arkansas River, the city heretofore discharged its untreated sewage. During the summer months both rivers carry little water, practically all of which is used for direct irrigation. The pollution of streams used for stockwatering and irrigation purposes by untreated sewage has become a serious health problem, affecting downstream users of both rivers. To comply with the findings of the State Board of Health regarding pollution of these rivers, the city decided to install sewage treatment works.

Choice of treatment

Widely varying seasonal conditions and the desire to keep the plant cost at a minimum made a satisfactory solution of the treatment problem difficult. For eight months of the year the average population is 40,000 and average sewage flow 8 m.g.d. For four months, starting June 1, the normal population is increased by a large influx of tourists reaching a peak of 75,000. The average summer sewage flow increases to 10 m.g.d. with short-period maximum flows up to 15 m.g.d.

The sewage is average domestic, and in a fresh condition when it reaches the plant site. Suspended solids average 225 p.p.m. during the daily period from 8 a.m. to 8 p.m. The average B.O.D. for the same hours is 180 p.p.m. About 65 per cent of the flow and 81 per cent of the organic load will reach the plant during the above mentioned period. Suspended solids and B.O.D. in the night flow are low.

Careful consideration of all factors indicated the necessity of high grade treatment during the summer irrigation season, with efficient primary treatment during the remainder of the year. Tentatively it was decided that the plant should be capable of removing 90 per cent of the suspended solids and 75 per cent of the B.O.D. during the irrigation season and 65 per cent of the suspended solids and 50 per cent of the B.O.D. during the remainder of the year.

Plant capacity was based on handling a maximum flow of 15 m.g.d. from a population of 75,000 for a three-month period yearly. For the remainder of the year the average design flow and population were considered as 10 m.g.d. and 60,000 inhabitants, respectively.

Trickling filters were considered but rejected because of the high first cost, the large area requirements and the proximity of the plant site to residences. The activated sludge process was rejected because of high first cost and the difficulty of adjusting the degree of treatment to the varying hourly and seasonal treatment requirements. The chemical precipitation process was chosen because it was considered to be the most economical, most efficient and the best adapted method of treatment suited to this particular problem.

Factors influencing plant design

In many respects the type of plant selected is distinguished from other similar plants in this country. Prior to the design of the plant careful investigations of the flocculation and sedimentation characteristics of the sewage were made to determine the proper detention periods to use in each case. These indicated that efficient chemical treatment for 12 hr. each day, from 8 a.m. to 8 p.m. during the irrigation season, would satisfy the complete treatment requirements. All chemical treatment studies were therefore based on the sewage load during the hours mentioned.

Flocculation was studied with and without pre-settling and it was found that pre-sedimentation was a valuable adjunct. During the above-mentioned daily period the hourly maximum suspended solids content in the raw sewage was approximately twice the average. The content varied hourly. Efficient and economical chemical treatment, in consequence, would require changing the rate of chemical dosage every few minutes to suit the varying sewage characteristics. Obviously in a large plant such a condition would be highly undesirable and would require continual manual attention. Pre-sedimentation before the addition of chemicals largely remedies this condition and results in more uniform solids removal with less chemicals. The suspended solids content following pre-sedimentation is almost uniform, with the maximum hourly content never exceeding the average by more than 20 per cent.

In general it was found that either ferric chloride or chlorinated copperas, when added to the settled sewage in dosages varying from 1.5 to 2.0 gr. per gallon, readily flocculated the sewage. Satisfactory flocculation in the laboratory was effected in a 20-min. mixing period with an addition of 2 gr. per gallon of ferric chloride. Slightly longer mixing periods reduced the amount of chemicals required.

Sedimentation studies indicated a minimum desirable detention period of 2 hr. Additional detention reduced the amount of small floc carried over. Stirring the settled sewage, without the addition of chemicals, proved beneficial and increased the overall removal of suspended solids.

Plant units and function

The principal units comprising the plant and a description of their function are as follows:

Two 27-in. dia. comminutors will shred all coarse material entering the plant thus simplifying the screenings disposal problem. A float control will permit intermittent operation. Grit removal will be accomplished in an 18-ft. square detritor designed effectively to wash the grit which will be used as fill around the plant.

The pre-settling clarifier will be 115 ft. in dia. by 11 ft., side water depth. Provisions will be made to skim the unit. At the maximum flow of 15 m.g.d. the sedimentation period will be 1.4 hr. Detention, at the 10.0 m.g.d. rate, will be 2 hr.

Two flash mixers of the elevating type, equipped with variable drive, will be installed in separate tanks, 12 ft. in diameter by 12 ft., side water depth. The units will be arranged for operation either in series or in parallel. Depending on the flow and method of operation the mixing period will vary from 1 to 3 min.

The two flocculator channels, each 30 ft. wide by 67 ft. 6 in. long by 13 ft. 9 in., side water depth, are provided with three rows of 12 ft. 6 in. dia. horizontal paddles arranged to rotate transverse to the direction of flow. The flocculation period, when operating in series at the 10 m.g.d. rate, is 30 min. Operating in parallel at the 15 m.g.d. flow the detention will be 40 min.

For final sedimentation there are two clarifiers, identical in dimensions to the pre-settling unit. These may be operated in parallel or in series. The detention period at the rate of 15 m.g.d. will be 2.74 hr., when the units are operated in parallel. At the 10 m.g.d. rate, with series operation, the detention will be 2 hr.

A multiple digestion system will be employed for treating the sewage solids. The primary digester will be 60 ft. in dia. and 22 ft., in depth, with a hopper bottom. It will be equipped with a center-guided dome-shaped gas holder having a storage capacity of 22,250 cu.ft. At the maximum anticipated gas production the holder will have an 8.4-hr. storage. The two units comprising the digestion system will provide a total minimum storage period of 60 days, at the maximum anticipated sludge loading.

For general chlorination purposes, and to produce the chlorinated copperas required for chemical precipitation, two solution-type chlorinators will be used each with a capacity up to 400 lb. per hr. Two chemical feeders will be used, one for lime and the other for ferrous sulphate.

A central control building, 30 ft. x 45 ft., with full basement and two floors has been provided. Sludge pumps and the heating and gas collection system will be located in the basement. Chlorinators, chemical feeders, chlorine storage and control switches will be on the first floor, while the second floor will be given over to office, laboratory and chemical storage space.

A complete sludge-and-garbage incinerator will be housed in a separate building 15 ft. wide, 25 ft. long and 41 ft. high. The plant will have a capacity of 4 tons per day (dry basis) of dewatered, digested sludge, plus 6 tons per day of milled garbage. Digested sludge will be conditioned with ferric chloride and then dewatered on a vacuum filter to reduce the moisture from 90 to 70 per cent, prior to discharge to the incinerator. The dewatered sludge will then be mixed with the garbage and fed into a circular multiple hearth furnace for incineration, using excess gas from the digesters to aid combustion.

Various methods of operation

The layout adopted is flexible and will permit several variations in treatment methods. During the early stages of plant operation much experimental work is planned to determine the relative efficiencies of different combinations. The following description will be restricted to the three operating schemes which should most satisfactorily solve the requirements.

Partial Treatment—Partial treatment will be used during the fall and winter months and presumably during the night flow periods the remainder of the year, when it is expected that chemical precipitation will be unnecessary. The usual primary treatment will be given, with the flow passing successively through the comminutor, the detritor and clarifier No. 1. The flow will then enter the flash mixer and flocculator No. 1 where surface aeration and gentle mixing will be effected. This flocculation will result in additional sedimentation of finely divided solids which will then be removed in clarifier No. 3. The effluent from clarifier No. 3 will pass through flash mixer and flocculator No. 2 where chlorine will be applied. Clarifier No. 2, receiving this flow, will act as a chlorination contact chamber.

Sludge from clarifiers Nos. 2 and 3 can be pumped directly to the multiple digestion system, or may be returned to the primary clarifier for thickening. All sludge from clarifier 1 will be removed to the digesters. Chlorination will be resorted to only during the irrigation season.

The removal from this sequence of treatment is expected to exceed that usually obtained in primary plants. The surface aeration, gentle flocculation and three-stage sedimentation will materially increase the B.O.D. and suspended solids reduction.

Series Operation—Chemical precipitation will be practiced with the units operated in series, during those intervals when complete treatment is required, and the flow approximates 10 m.g.d. or less.

Primary treatment will first be afforded by the shredding, grit removal and preliminary sedimentation. Settled effluent will be dosed with a conditioning reagent plus return sludge from clarifier No. 2 after which it will enter flash mixer and flocculator No. 1. The flocculated liquid will then be settled in clarifier No. 3. The effluent from clarifier No.3 will flow to flash mixer No. 2 where it will be dosed with the requisite chlorinated copperas. Further flocculation will be provided in flocculator No. 2 and the flow will then pass to clarifier No. 2 for final sedimentation. The overflow from this unit will be suitable for final disposal.

The conditioning reagent added ahead of flash mixer No. 1 may vary from time to time. Experiments indicate the advantages of pH adjustment at this point in order to obtain the optimum chemical reaction at a minmum dosage. The feasibility of adding an inexpensive low-grade base-exchange pulverized material at this point to increase the removal of soluble organic nitrogenous solids will be studied. A low pH will be maintained previous to the addition of the base-exchange material and a recirculating load of the precipitated clays to the point of original addition is planned.

Eventually, excess sludge from clarifiers Nos. 2 and 3 will be returned to clarifier No. 1 for thickening. All sludge will then be pumped to the digestion system for stabilization.

It is anticipated that this method of operation will produce almost complete removal of suspended solids and a high B.O.D. reduction. The plant effluent should be sparkling clear and entirely odorless. Chemical costs should be reasonable. Full advantage will be taken of return-sludge possibilities, efficient flocculation and sedimentation as a means of keeping chemical consumption at a minimum.

Parallel Operation—When the sewage flow materially exceeds 10 m.g.d. it is expected that chemical precipitation can be most advantageously carried out by operating the final units in parallel.

As in the series operation described above, the sewage will be given primary treatment. This will be followed with the proper dosage of chlorinated copperas. One-half the flow will then enter the No. 1 flash mixer and flocculator and the other half will pass to flash mixer and flocculator No. 2; each portion will then flow to clarifiers No. 3 and 2, respectively, for final sedimentation. The final effluent will be discharged to the outfall canal for disposal in the irrigation ditch. Sludge from clarifiers Nos. 2 and 3 will be returned to clarifier No. 1 for thickening and mixing with the primary sludge previous to digestion.

This scheme of operation is somewhat simpler than series treatment and will probably not produce as high grade an effluent for the same unit cost. However, the degree of treatment possible should amply satisfy all public health requirements.

Unit plant costs low

The Colorado Springs plant is being constructed as a PWA project with a grant only, the city furnishing its share of funds by a bond election. The plant costs, based on actual bids, are shown in the accompanying table.

A comparison of the plant costs with similar sized projects throughout the country shows it to have a unit cost that is little (if any) more than the average primary treatment plant and far less than most of the chemical precipitation plants. The low costs are the result of careful planning to effect an economical grouping of individual units to achieve the desired flexibility of operation. Special attention was given to the design of concrete structures to simplify construction, thus keeping costs at a minimum. Rectangular concrete conduits for the influent and effluent will be built, the top slabs of which will furnish walks to the various units. Such details were found to have an important effect on low first cost.

Equipment contracts, exclusive of the units required for the incineration plant, were let prior to calling for bids on general plant construction. General contract bids were entertained on the basis of a plant providing sludge and garbage incineration. The alternate, with incineration, was chosen because of the expected advantages in this method of disposal.

CHEMICAL PRECIPITATION PLANT WITH SLUDGE DRYING BEDS

Mechanical equipment	$60,424.48
Materials furnished by city.....	13,000.00
General plant with sludge beds..	134,836.90
Total plant cost	$208,261.38
Cost per m.g.d., design capacity.	$13,884.10
Cost per capita, design population	2.78

CHEMICAL PRECIPITATION PLANT WITH INCINERATION

Mechanical equipment	$60,424.48
Materials furnished by city	13,000.00
General plant with incineration.	168,804.75
Total plant cost	$242,229.23
Cost per m.g.d., design capacity.	$16,148.60
Cost per capita, design population	3.23

The general plant contract, including sludge and garbage conditioning, and incineration features, was let to the Chambers Construction Co., of Lincoln, Neb., based on their low bid of $168.804.75.

Acknowledgment is hereby made for the assistance and cooperation of Earl L. Mosley, city manager, Frank O. Ray, city engineer, and George M. Bull, acting state PWA director, and also of B. V. Howe, state sanitary engineer, State Board of Health, for furnishing sanitary data regarding stream conditions.

Improving Performance Of the Trickling Filter Process

Attention is focused on schemes for increasing the efficiency of trickling filters in research directed toward the refinement and further development of biological methods of sewage treatment

By Willem Rudolfs
*Chief, N. J. Sewage Experiment Station
New Brunswick, N. J.*

THE SEARCH for a dependable, easily operated method of biological sewage treatment, prompted by certain weaknesses in existing methods, has been progressing quietly for many years. Considerable experimentation has been directed toward the improvement of the trickling filter process against which the chief objections have been the relatively low rates of application and the fly nuisance. The following briefly summarizes the results of this research and indicates in which direction progress has been made as well as possible further development.

Utilization of pre-aeration or partial activation before treatment on trickling filter beds has been demonstrated to be of considerable value at Decatur, Ill., and elsewhere. Complete activation followed by trickling filters has been employed by Kessener at Knocke (Belgium) and in England at several places, giving very high purification results with almost complete removal of ammonia at Knocke. In the Lippe distrct Prüss and Blunk designed a trickling filter followed by activated sludge, requiring only 0.15 cu.ft. of air per gallon and an aeration period of 20 min. The advantages in putting the aeration unit behind the trickling filter are low air consumption, ease of operation (because variations in flow have been equalized by flowing through the plant), complete oxidation and low suspended solids in the effluent at all times.

The studies on an artificial stream by the Public Health Service at Cincinnati, the description of sewage treatment by stream flow aeration by Jenks and Levine, as well as the earlier experiments on thin sewage films with introduction of air by mechanical means (fan by Forst at Brooklyn, paddles by Haworth at Sheffield, and many others) fall in the category of research for a dependable method of purification, where sometimes a part of the effluent or a part of the sludge was returned for increasing the rate of oxidation.

Vacuum filter studies

An experimental plant of a vacuum trickling filter type at Soest, Germany, has been operated by Blunk. The trickling filter, tightly sealed, through which air is drawn from the bottom to the top is stated to produce results equal to the large plant trickling filters, although the flow is from 6 to 7 times higher. A similar unit has been in operation for some time at Hilversum, Holland. A somewhat modified closed "vacuum trickling filter" with a series of pipes and other provisions to draw heated air through the body of the filter, has been installed at Zurich.

On the basis of experimental results, Baumont suggested in 1923 to return a part or all of the effluent from a trickling filter to the influent. Halverson and Smith in their "aero-filter" maintain a high continuous flow through the filter by returning a portion of the plant effluent through the filter during periods of low sewage flow. It is stated that forced draft ventilation through the filter at those seasons of the year when the temperature of the sewage is about the same as the air improves the efficiency of the filter. They maintained at all times average flow rates of not less than 20 m.g.d. and produced 84-85 per cent B.O.D. reduction of Minneapolis sewage during the months of January and February. Jenks in his "bio-filters" repeatedly passes the sewage at high rates through shallow trickling filters, producing a B.O.D. removal between 70 and 80 per cent at rates varying from 4 to 24 m.g.a.d. This reduction is based upon the B.O.D. of settled sewage, whereas the reductions reported by Halverson and Smith apparently are based upon the raw sewage which varies from 200 to 700 p.p.m. B.O.D. The depth of the Halverson-Smith filters is 10 ft., whereas Jenks' filter is only 3 ft. With simple treatment (in one bed) intermediate results can be obtained, while with two stage treatment higher purification can be accomplished.

Further advances suggested

All types of new trickling filters, from vacuum filters to aero and bio-filters, are an improvement of the old system. There is reason to believe that refinement and modification of any or all of the systems will allow further advances, such as by splitting the functions of a trickling filter (straining, reduction and oxidation), returning seed material in proportion to the load applied and groups of organisms desired, variation in size of mediums in relation to suspended solids removal, distribution of sewage and air, and stimulation of desirable and retardation of undesirable growth complexes. With the improvement and diversification of filter functions the rate of sewage stabilization will be increased and fly nuisances prevented. Undoubtedly the newer methods will be more complicated from an operation standpoint, but this does not necessarily mean that they will be more expensive.

Designing the Longest Fixed Steel Arch

Main span of Henry Hudson Bridge at New York is 800 ft. long, a new record for hingeless steel arches, and consists of two box plate-girder ribs 12½ ft. deep and 3½ ft. wide carrying a 42-ft. deck roadway

By D. B. Steinman and C. H. Gronquist,
Robinson & Steinman, Consulting Engineers, New York, N. Y.

THE MAIN 800-ft. span of the Henry Hudson Bridge at present under construction over the mouth of the Harlem River from Inwood Hill Park in Manhattan to Spuyten Duyvil in the Bronx, New York City, is the longest span fixed arch as well as the longest span plate girder arch built to date. It consists of two double girder arch ribs of silicon steel 12½ ft. deep and 50 ft. apart with a rise of 120 ft. which support a superstructure composed of spandrel columns, continuous fascia girders, and a deck on a maximum grade of 2½ per cent. Steel towers, architecturally a dominant feature at either end of the arch span, are flanked by two 300-ft. continuous steel girder viaducts of 60-ft. spans, and concrete frame and wall approaches. The total length of steel is 1,530 ft. and of the entire structure is 2,000 ft. The structure's single deck will have a 42-ft. roadway and a 4-ft. sidewalk. The elevation of this roadway at the center of the bridge will be 160 ft. above high water, to provide an underclearance at that point of 142.5 ft. above high water.

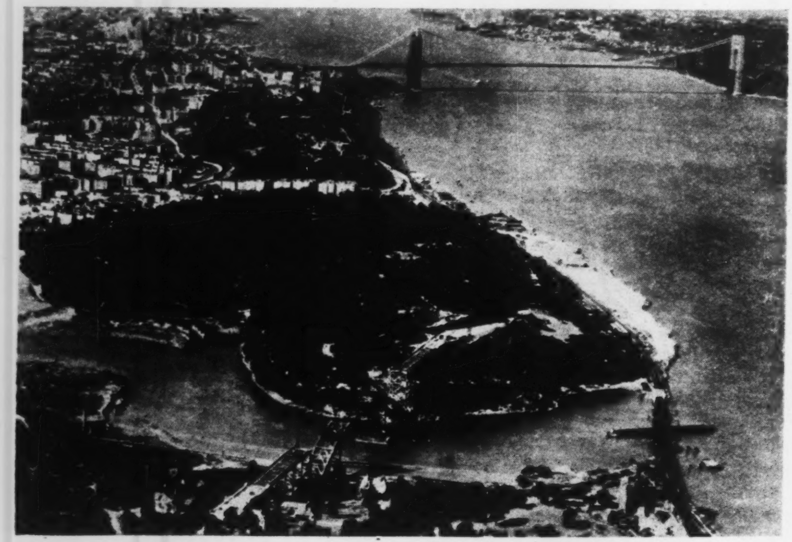


FIG. 1—SPANNING the Harlem River near its junction with the Hudson, the steel arch of the Henry Hudson Bridge forms a link in the parkway that connects with Riverside Drive at Dyckman St., identified in the view by the row of apartment houses set in the trees of Inwood Hill Park.

Comparative designs of two-hinged and hingeless arch types showed little, if any, difference in economy between them. The two-hinged arch, however, required the heaviest sections at the quarter point whereas the arch designed for fixed-end conditions provided ribs with a constant minimum section for three-fourths of their length, with a gradual increase in section toward the skewback for a distance of 100 ft. Comparison of quantities indicated somewhat more metal in the two-hinged ribs. For the hingeless arch, anchorage of the skewback to the foundation is simple, being obtained by eighteen 3-in. diameter upset high-tensile steel bolts. However, the 9 ft. 2 in. skewback sections with a 4-in. base billet 20 ft. by 6½ ft. in area that are required, bear on a heavy grillage 6 ft. deep with a 4-in. base billet 20x14 ft. in area. This is much larger grillage than would have been required with the two-hinged arch, but it eliminates the expensive pin detail that the two-hinged arch requires. The greater stiffness of the fixed arch under live load is distinctly advantageous, and for this and the reasons cited above, the fixed arch type was chosen.

A crossing at Spuyten Duyvil, overlooking the Hudson River with the Palisades in the distance, was first proposed thirty years ago. A complete design for a concrete arch of 725-ft. span, to carry rapid transit as well as highway traffic, was prepared by the New York City department of bridges in 1907, and studies for a steel arch design were also made at that time. More recently a design for a steel arch with a single deck, to accommodate six lanes of highway traffic, was proposed.

The bridge now under construction is the principal structure of the Henry Hudson Parkway which will provide an extension of the West Side elevated highway and Riverside Drive to connect with the Saw Mill River Parkway and the Westchester County parkway system. The Henry Hudson Parkway will begin as a steel viaduct near 200th St., the present terminus of Riverside Drive, will cross over Dyckman St. on a reinforced concrete arch, and will then run for about one mile along the west side of Inwood Hill Park to the Harlem River crossing. North of the Henry Hudson Bridge the parkway will pass through Riverdale to Broadway near 254th St. and from there through Van Cortlandt Park to the Saw Mill River Parkway. Several concrete structures required to avoid grade crossings are now under construction along this route. The section through Van Cortlandt Park, or the Saw Mill River Parkway Extension, is being constructed by the state department of public works.

The Henry Hudson Parkway is being built by an authority set up by the New York state legislature as an auxiliary to the New York City department of Parks. Bonds in the amount of $3,100,000 which have been marketed by the authority to provide funds for the construction, will be redeemed by tolls. Because of difficulties in financing a six-lane parkway, the present plans are for a four-lane parkway and bridge with provision for the future addition of three lanes on the parkway and a future upper deck on the Henry Hudson Bridge after the justifying traffic is developed or demonstrated.

General design considerations

An attempt was made in fixing the proportions of the various parts of the bridge to achieve a maximum unity of design. The viaducts were regarded as a continuation of the arch superstructure, 30-ft. panel lengths being used throughout the structure except over the main towers, and the depth of viaduct girders and arch fascia being made the same, to carry through an unbroken deck line. The transverse bracing in the arch spandrels, main towers, and viaduct bents, of simple X-type with the lower strut omitted to give a portal effect, was also given unity by maintaining a constant panel height over the length of the bridge. The use of continuous viaduct girders and of continuous framing of the arch fascia girders made it possible to eliminate longitudinal bracing, and concentrated expansion at the towers. With the viaducts fixed to the concrete approaches, only four expansion joints are required, one on each side of the two towers.

The deck consists of an 8-in. reinforced concrete slab supported on longitudinal steel stringers framed into the floorbeams. The four-foot sidewalk on the west side of the bridge is separated from the roadway by a steel curb which also acts as a stringer. The steel curb on the east side is connected at intervals to the fascia girders and likewise performs the function of a roadway stringer. The fascia girders, surmounted by low pipe rails, serve as guard rails. The fascias and the floorbeams frame into short 14-in. H-sections supported on the spandrel columns, these H-sections being intended to provide attachment for the columns of the future deck. The 77-in. fascia girders are continuous by virtue of their end framing and splicing, and are considered as the chords of the nominal top lateral system which delivers a wind reaction to each main tower.

Plated spandrel columns

The spandrel columns are made up of four angles and two web plates connected by a 12-in. wide-flange beam diaphragm and two cover plates from which 11x26-in. oval holes are cut to permit access to the inside of the section for painting. This type of member is used in preference to a laced column on the basis of appearance and greater ease of maintenance. The longitudinal width of the columns varies from 36 in. in the case of the longest, 108 ft. high, to 24 in., except for those at the center of the arch which consist merely of the extension of the 15-in. channels in the arch rib diaphragms. The width of the longer columns was made sufficient to obviate the necessity of intermediate longitudinal bracing; the reduced width of the shorter columns was dictated by considerations of appearance. The top and bottom connections of the columns are designed to act substantially as hinged ends by concentrating bearing area at the center of the column and connecting the flanges to relatively flexible plates on the arch ribs and to the unstiffened flange angles of the fascia girders. By making these connections hinged in effect, the temperature moments, which would be produced in fixed columns due to the expansion of the deck steel, are practically eliminated.

The arch rib lateral system consists of a K-bracing system in the plane of each flange. The laterals are made up of four angles and a web plate to form an I-section, while the struts, two 15-in. channels laced, are trussed together to provide a stiff transverse member between the ribs at every panel point. The K-bracing system was considered desirable by reason of its stiffness, and also because the participation stresses due to the shortening of the ribs under load are negligible, since the lateral members in running from one rib to the other return to the same panel point, and change in rib length merely causes bending of the strut, which can usually occur without producing appreciable stress.

The arch ribs were designed to be cambered for final dead load, and the three-centered arch axis was laid out to follow closely the final dead load polygon, so that flexural stresses in the arch ribs for dead load are slight. Cambering of the ribs eliminates the dead load rib shortening stresses which must otherwise be provided for. In the estimate of dead load there is included an allowance of 3200 lb. per ft. of rib for the future upper deck.

Live load assumptions

The live loading assumed for this span, the floor system of which was designed for H-20 loading though trucking will not be permitted, is 50 lb. per sq.ft. of roadway surface, including impact, and 30 lb. per sq.ft. of sidewalk, for both present and future decks. This amounts to a total of 2200 lb. per ft. of rib on the east side, of which 950 lb. is for the future deck. The total calculated live load deflection of the center of the arch for full span loading is 1½ inches.

The arch ribs were designed for a temperature change of ± 60 deg. F. from a normal of 55 deg. F., and also for a possible error in span of ± 1 in., equivalent to a temperature change of ± 15 deg. F. The moments produced by temperature change in a fixed arch are of considerable magnitude. The computed temperature deflection is ± 6 in. from normal position.

In computing the live load stresses, the arch reactions, referred to the elastic center, and including the effect of live load rib shortening were first obtained for a unit load at each panel point. The reactions were then used to compute the moments about the kern points which give directly the influence line values for maximum total flange stress.

Computations of the deflection correction, or the stresses due to the change in arch rise produced by live load and temperature deflection, which was neglected in determining the foregoing stresses, show that these secondary deflection stresses do not for this arch exceed 10 per cent of the above combined dead load, live load, and temperature stresses. Since they occur in this magnitude where the wind stresses are small, they may be safely neglected.

Wind stresses in the arch ribs and lateral system were computed for assumed wind loads of 30 lb. per sq.ft. on 1½ times the vertical projection of the structure, and 200 lb. per ft. of bridge on both decks acting on the live load. Wind stresses in the ribs were obtained for two loadings, full-span and half-span loading. The method of computation was by resolving the stresses in the laterals into horizontal and vertical components acting on the ribs and lateral struts. The overturning effect combined with these components gave horizontal and vertical forces for which the rib stresses were computed as for live load. The design specifications permitted a 25 per cent increase in stresses to include wind, and as wind stresses even near the skewback were less than this amount, wind did not govern the rib design except in the case of the skewback anchorage details.

Allowable stresses on the silicon steel of the arch ribs for the combination of dead load, live load, temperature, and error were: tension 24,000 lb. per sq.in.; axial compression $\frac{24{,}000}{1 + 9000\ \frac{L^2}{r^2}}$; compression, axial plus bending, 24,000 per sq.in., with axial compression not to exceed two-thirds of the total stress and L/b not to exceed 10.

Rib makeup

The 148-in. silicon steel web plates of the arch ribs have a constant thickness of 1 in. throughout. This thickness was required to make the critical buckling stress equal to the yield point for direct compression or bending. The shearing stresses in the rib are small.

The minimum and typical rib section consists of two 150-in. silicon steel girders each composed of the 148x1 in. web plate, eight flange angles 8x8x$\frac{11}{16}$ in., and two cover plates 22x$\frac{1}{2}$ in. The maximum section which occurs adjacent to the skewback section is made up of two girders each composed of the 148x1 in. web plate, four flange side plates 24x$\frac{1}{2}$ in., eight flange angles 8x8x1 in., and two cover plates 22x1$\frac{1}{2}$ in. The 9 ft. 2 in. skewback section is 20 ft. deep, the flange angles running through at the 12 ft. 6 in. depth but with double webs extended and braced as diaphragms to provide generous depth to fix the hingeless arch.

Radial diaphragms and outside stiffener angles occur at the third points of each arch rib panel and at the columns, to stiffen the double girder transversely. A laced double girder rib was adopted in preference to a box girder section because of the economy and greater accessibility of the laced section. The girders are laced in three planes, at the outer flange angles and at the center by connection to 6x6x$\frac{1}{2}$ in. silicon steel angles which also form part of the main rib section. The lacing through the minimum section is single lacing made up of 10 in. 20-lb. channels at the flanges connected by six 1 in. diameter rivets and 6x3$\frac{1}{2}$x$\frac{3}{8}$ in. angles connected by four $\frac{7}{8}$ in. diameter rivets at the center of the rib. This lacing has a shear capacity of one per cent of the total main section at 24,000 lb. per sq.in. with the rivets computed at 13,500 lb. per sq.in. in shear. The lacing through the heavy sections in the 100 ft. near the springing line is single lacing made up of 15 in. 35-lb. channels at the flanges connected by eight 1-in. diameter rivets and 7x4x$\frac{7}{16}$ in. angles connected by four 1-in. diameter rivets at the center of the rib. This lacing also has a capacity of one per cent of the main section, at a distance of 10 ft. from the springing line where the lacing begins.

The arch rib splices are located just beyond the river side of each column. The splice material is designed to splice 50 per cent of the rib section, the joint being milled to bear. The splice at the center and closing point of the arch was designed to splice for the entire section to allow for adjustment in closure. One inch diameter rivets were used in the arch rib main material and details and in the lateral connections. In the splices 1$\frac{1}{8}$-in. diameter rivets were employed; $\frac{7}{8}$-in. diameter rivets were used elsewhere.

The 30 ft. long rib sections, weighing approximately 40 tons, were milled after accurate measurement of their length and of the angle of the ends had been made. The rivet holes at the joints were subdrilled to $\frac{13}{16}$-in. diameter and later reamed to full size 1$\frac{3}{16}$ in. diameter with the rib sections assembled and abutting ends in contact. Accuracy of layout was checked by measurement of length, offsets from the chords, and closing angles of successive 200-ft. shop assemblies.

Foundations

All foundations were carried to rock. At the south skewbacks located at the edge of the river, rock is just under the surface and is of the familiar Manhattan or Hudson schist with a considerable content of mica. The north skewbacks are 300 ft. from the river. The maximum depth to rock at this location is 33 ft. below high water or 37 ft. below the surface. This rock is Fordham gneiss. The rock rises rapidly from the river, providing natural abutments for the arch.

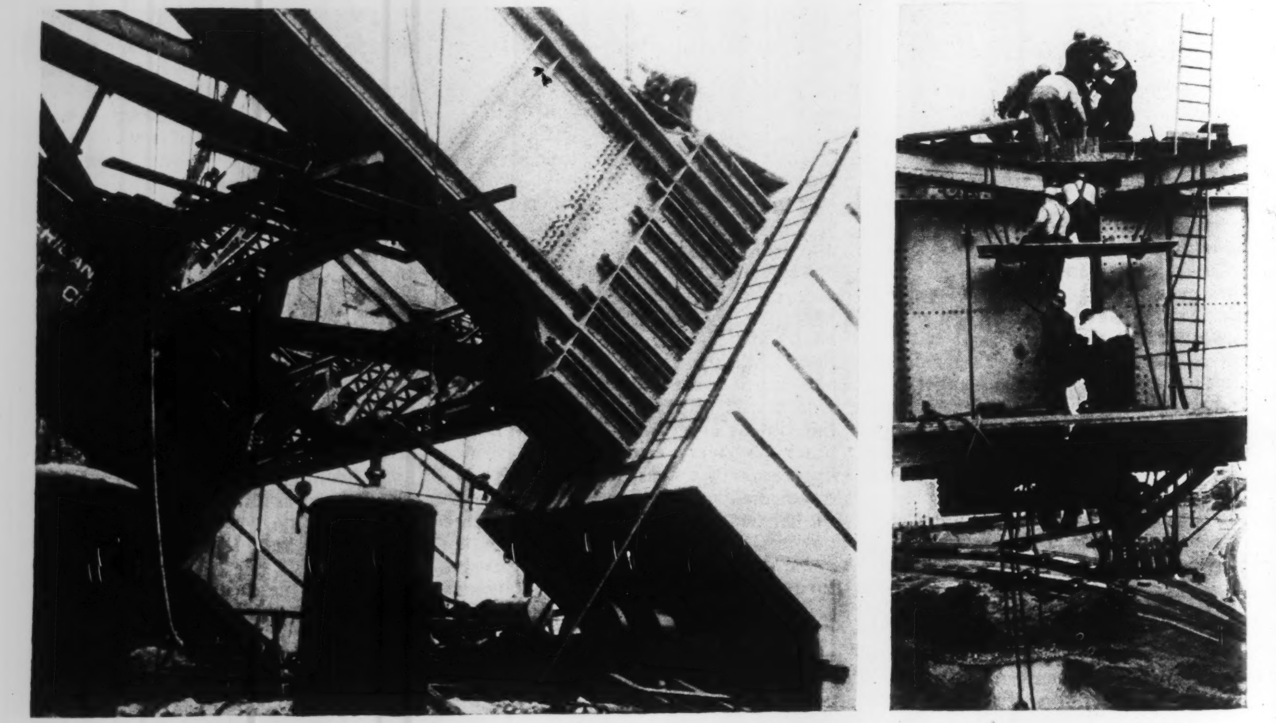


FIG. 3—FIXITY where pin joints usually occur in steel arches. Left: Skewback anchored to a buried grillage by 18 high-strength steel bolts 3 in. in diameter. Right: Jacking an opening at the crown into which a 2½ ft. closure section was riveted.

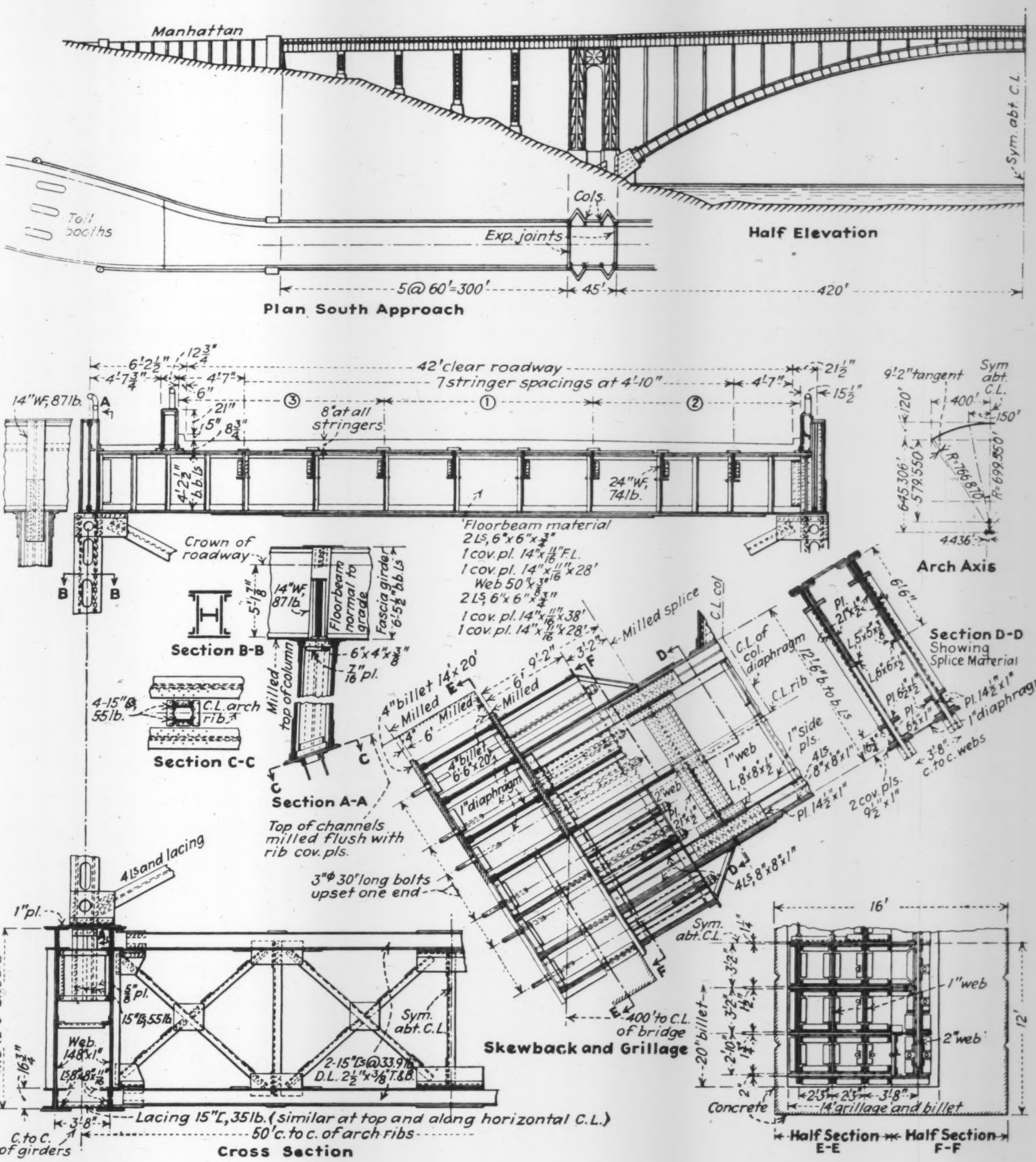

Manhattan
Sym. abt. C.L.
Toll booths
Cols.
Exp. joints
Half Elevation
5@60'=300'
45'
420'
Plan South Approach
42' clear roadway
7 stringer spacings at 4'-10"
14" WF, 87 lb.
8" at all stringers
24" WF, 74 lb.
Floorbeam material
Crown of roadway
Section B-B
Section C-C
Section A-A
Top of channels milled flush with rib cov. pls.
3"φ 30" long bolts upset one end
Milled splice
C.L. col
C.L. of col. diaphragm
C.L. rib
1" diaphragm
1" web
Section D-D Showing Splice Material
Arch Axis
9'-2" tangent
Sym. abt. C.L.
4 Ls and lacing
Sym. abt. C.L.
Lacing 15" [, 35 lb. (similar at top and along horizontal C.L.)
50' c. to c. of arch ribs
C. to C. of girders
Cross Section
Skewback and Grillage
400' to C.L. of bridge
16'
1" web
2" web
12'
Concrete
14' grillage and billet
Half Section E-E
Half Section F-F

FIG.2—STRUCTURAL DETAILS of the 800 ft. hingeless arch span of the Henry Hudson Bridge.

Individual piers are used for each rib. The north pier footings are 25 ft. by 75 ft. in plan while the south pier footings are smaller since rock is near the surface and rises more quickly than on the north side. The pier footings are stepped. The north foundations were constructed in steel sheet pile cofferdams. The south foundations are on shore except for a small part of each which was built behind a puddle wall cofferdam with timber sheeting. The concrete main-pier superstructure extends to a height of 37 ft. above high water. The architectural design of this portion of the pier suggests the carrying back of the lines of thrust of the arch which is accented by the use of 6-in. grooves in the concrete thrust block. The shape of the pier was further determined by the necessity of accommodating the plan of he steel tower.

The main towers supply an architecturally needed element of mass at the ends of the arch. They are of steel rather than the usual concrete construction and are 115 ft. high to the roadway level. Each tower is composed of four bents similar to those on the arch, but with columns only 20 in. deep, since they are braced longitudinally. The outer and inner columns are braced in pairs with X-bracing; the inner columns are also connected at the deck by a semicircular arch portal and bracing in the form of the Cross of St. George. To give the tower greater lateral depth and a distinctive form, four buttress columns, which also serve to support triangular recesses in the sidewalk, are K-braced to the pairs of outer and inner columns. Cast steel rockers on each side of the fixed main towers permit temperature movement of the arch and viaduct decks.

Viaducts

The cross-section of the 300-ft. viaducts is similar to that of the arch, the continuous girders taking the place of the arch fascia girders. Design of the five-span, 78-in. girders for continuity, even though the roadway lies on a vertical curve, produced reduction of flange area and increase in rigidity. Elimination of longitudinal bracing avoided breaking the viaduct into separate braced units, thus making for a more harmonious and unified structure.

The viaduct columns are spaced 50 ft. apart transversely as on the arch, and also have similar though heavier X-bracing. All columns are made up of two 21 in. 112-lb. wide flange beams spaced from 3 to 5½ ft. apart, depending on their length, and laced with double angle lacing so arranged as to form a perfect diamond pattern. The long columns, being fixed at the base and hinged at the top, are designed for the bending produced by temperature expansion of the girders. Several of the shorter columns, too stiff to bend under moderate stress, require a rocker detail at the base, obtained with a curved slab. The foundations consist of individual octagonal shaped pedestal piers under each column. These piers are founded on rock.

Massive concrete abutments act as the fixed point for the viaduct. These piers are about 8x15 ft., and one on the northwest side is 50 ft. in height. The piers at the end of each girder support the deck between and are connected across the front by a curtain wall. The concrete approaches extending beyond the abutments are built as a series of two-span frames 15 ft. on centers which support and are enclosed by 12-in. curtain walls. The frames are composed of an outer column and beam, the inner end of the beam being simply supported on an independent center column to relieve the wide structure of high temperature moments. The rear portions of the approaches are simple retaining walls, to hold earth fill. The total length of the north approach is 230 ft. and of the south 240 ft. Foundations are in general carried to rock except at the end of the north approach where the high ridge of rock on which the remainder of the approach lies, drops off too steeply to have made it practicable to carry the footings to rock. The sloping character of this rock on the north side led to a decision to enlarge the bases under several of the east columns.

The sole member of the Henry Hudson Parkway Authority is Robert Moses, park commissioner of New York City. The remaining officials of the Authority are W. Earle Andrews, chief engineer and general manager, Harry Taylor, secretary, and W. H. Latham, park engineer. The designing engineers for the Henry Hudson Bridge are Robinson & Steinman, consulting engineers, New York, and the consulting architect is Aymar Embury II, who was assisted by William Jensen. The engineers in charge of design, preparation of contract drawings, and specifications for the Henry Hudson Bridge for the firm of Robinson & Steinman were C. H. Gronquist, J. London, R. M. Boynton, and M. Birnbau

The substructure contract for t Henry Hudson Bridge, awarded in Ju 1935 to Thomas Crimmins Contract Co. for $272,668, is completed. T superstructure contract was awarded the American Bridge Co. for $865,2 The erection of the viaducts, main tow and arch ribs has been completed, a the erection of the arch deck is well a vanced. Completion of the bridge scheduled for this fall.

Steel Water Tank on a Tall Tower Moved 350 Ft. to a New Location

Base rigidly braced with timbers—Counterweights added to this framework to offset 50-mile wind pressure increases load from 65 to a total of 160 tons

By Milton C. Shedd
Wm. P. Neil Company, Ltd., Los Angeles

RECENTLY, a 75,000-gal. water tank on a steel tower near Los Angeles harbor had to be moved to make way for new buildings. As the new site for the tower was only 350 ft. from the original position, decision was made to move the entire structure bodily. This was accomplished successfully although in the process some unusual requirements had to be met including provision for the horizontal load of a 50-m.p.h. wind.

The total height of the structure is 125 ft.; the legs of the tower are inclined so that the base is in the form of a square with 35-ft. sides. The tank was erected in 1916 and was damaged in the 1933 Long Beach earthquake to such an extent that considerable strengthening was required. Bent flanges on the legs of the tank were straightened cold and new steel totaling about 11 tons in weight was installed piece by piece, partly by welding and partly by riveting to original members.

As the tank proper presents an area of 25x40 ft., centered 100 ft. above the ground, on which wind pressure is effective, it was thought necessary to make the structure safe for a wind of 50 m.p.h. that might spring up during the moving. This called for counterweights at the base of the tower and accordingly 20 tons of sacked cement was loaded on a platform built up for the purpose at the base of each of the four legs. The original weight of the tank was 65 tons and the total weight when ready for moving including counterweights, bracing framework, timbers, etc., was about 160 tons.

Preparations for moving included bolting on to each leg 18 in. above the base a pair of 15-in. channels 12 ft. long. Two pairs of 12x16-in. skid timbers 40 ft. long were placed under these channels and bolted in place. The skids, when ready for moving, rested on solid steel rollers 2 in. in diameter and 30 in. long. Two sets of rollers, with about 20 rollers in each set, were spaced 6 in. apart under each tower leg. The rollers, in turn, were supported on runways two ft. wide made up of 6x12-in. timbers laid on the flat. The bed for these runways consisted of a mat of railroad ties laid solid. Only 60 lin.ft. of the runway was laid at a time to prevent blocking the highway; runway timbers and ties beneath them were taken up and moved ahead as the tank advanced.

Prior to doing any jacking, 8x12-in. cross timbers and 6x12-in. diagonal members were bolted to the tower legs to form a rigidly braced base. Platforms were built over the skid timbers for the counterweights. The tank structure and the base bracing system was found to be so rigid that if one leg was jacked up ½ in. higher than the others, one-half the total load of the tank immediately was concentrated on this leg. Every effort was made to keep the runways exactly level but some settlement did occur. Several times the tank was found to be teetering on two legs. The soil is sea sand and shells in which moisture begins to appear a few inches below the surface.

When the structure was ready for moving two sets of jacking blocks were placed under the ends of the 15-in. channels that had been bolted to each leg. Two 15-ton screw jacks were used at each of the eight jacking blocks. After these had been forced to a firm bearing the entire load was raised 18 in. to permit the mat of railroad ties and the runways to be laid beneath the skids. While the load was jacked up wedges were kept tightly driven under the base plates of the columns as a matter of

MOVING AN ELEVATED TANK at Wilmington, Calif., required the use of heavy counterweights to prevent danger of overturning in a 50-mile wind.

safety. At each corner 6x12-in. pry timbers were used at the start to indicate whether the jacking process was really raising the tank or just forcing the footing into the ground.

LINE
MATSON LINE

COUNTERWEIGHTED BASE of the tower. The pry timbers were telltales to indicate whether the load was being raised or whether the jack footings were being forced into the ground.

Motive power was provided by a truck with a winch attachment anchored directly ahead of the tank. A four-part line was attached at the rear of the base frame. For a straight-ahead pull the load on the pulling cables appeared to be about 5 tons and this was found to be about doubled when a 25-mile head wind sprang up.

Several times in the course of the move the direction of travel was changed by tapping the rollers to an angle with the runways. Part of the way the path was on a curve of about 300-ft. radius on which the tank was turned through an angle of 90 deg. When turning, additional power was applied by a light truck using a pair of three-part blocks.

The actual moving time in traveling the 350 ft. was about 20 hours. About 15 hours was used in raising and placing the runways and 12 hours for lowering the structure to the new foundation. The crew consisted of fourteen workmen including the truck drivers. The writer, assisted by Carl Hansen, job superintendent, directed the work for the contractors, Wm. P. Neil Co., Ltd., of Los Angeles.

Cross-Connection Hazards at Sewage Disposal Plants

By C. E. Keefer
Principal Assistant Engineer, Bureau of Sewers, Baltimore, Md.

DURING the past ten years there has been a marked advance in the design of sewage treatment works but in one particular—the elimination of cross connections—designers have not been overly thoughtful. This condition was apparent to the writer during a recent inspection of several sewage disposal works in the East and Middle West. While the plants appeared in general to be carefully planned and well constructed, cross-connections between water mains and pipes containing sewage and sludge were observed at several installations. These cross-connections consisted of water pipes connected to the stuffing boxes of centrifugal pumps handling sewage and storm drainage, to water seal tanks used in conjunction with venturi meters for measuring the flow of sewage, and to the heating coils in sludge tanks.

It is quite true that a somewhat unusual series of events must occur if these cross-connections lead to the contamination of a water supply system, but the fact is that such happenings have occurred. On the record of one of the plants inspected in Illinois there is the instance of a water supply pipe connected to the stuffing box of a pump being contaminated.

The solution of the problem is comparatively simple: Under no circumstances should water pipes be cross-connected with other pipes or devices, through which sewage or contaminated liquids flow. Where water under pressure is required, it can be obtained by installing a small pump, the suction piping of which is carried to a tank. This tank should contain a float valve for controlling incoming water and should also be provided with an overflow. The overflow is located below the outlet of the float valve so that it will be impossible for water in the tank to be sucked back into the main due to an unexpected failure of water pressure. A tank similar to that described can often be used without a pump where the pressures required are not great.

Pressure grease cups, connected with the packing glands of centrifugal pumps, can sometimes be substituted for water seal piping. Grease cups should not be used, however, without the approval of the pump manufacturer. It is not advisable to use sewage as a water seal by connecting the packing glands and the pump casing with a small pipe, for the supply pipe will soon become clogged with suspended materials in the sewage.

Where seal tanks are provided between venturi meters and register-indicator-recorders, it is desirable to connect these tanks with a source of water supply under sufficient pressure so that the tanks, the piping to the meter and the pressure chambers or piezometer rings around the meter can be flushed out periodically. A satisfactory way of doing this is to use a small pressure storage tank (preferably one with a gauge glass, to tell when the tank is empty) connected with a source of compressed air. An opening or a funnel with a valve is provided in the top of the tank so that water can be introduced. By closing this valve and admitting compressed air, the water in the tank can be used for flushing the seal tanks and the pressure piping to the venturi meter.

While it is difficult to formulate rules to meet all conditions, it seems safe to say that state boards of health should not, under any condition, permit the installation of cross-connections at sewage pumping stations and treatment works.

FORT PECK DAM TODAY—III

Tunnel Enlargement and Lining

Ingenious adaption of top heading and bench method overcomes difficult problem of enlarging the large bores in treacherous shale—Lining follows close behind excavation

H. W. Richardson
Associate Editor, Engineering News-Record

ENLARGING and lining of the four Fort Peck diversion tunnels is probably the most interesting and difficult tunneling work under way in the country. Driving against time—it is planned to finish the tunnels for diversion next July—forces the job ahead at a rapid pace. Driving through treacherous, blocky Bearpaw shale is difficult, exacting and painstaking work, where every square inch of exposed surface must immediately be given three coats of bituminous sealing solution to check disintegration, and where every square foot of roof must be caught up and blocked at once to hold the ground in place.

Driving operations are difficult enough in themselves, but top heading and bottom bench excavation must be carried ahead, along with setting of steel ring beams, welding of ring joints, placing of reinforcing bars up to 1½ in. sq., pouring of invert concrete, placing the sidewall and arch lining and grouting the crown. All of these operations must be carried on simultaneously in each of the four tunnels without interference with each other. In addition there is the work of excavating and lining of the eight control shafts that tap the tunnel about mid-point.

The multiplicity of operations invites confusion; there is every chance for a glorious tie-up, for damaging interference and for a break in the working schedule. Yet, surprising enough, the tunneling work is progressing rapidly and smoothly, without confusion and without interference. It is a fine example of a splendidly-organized job, where ingenuity, skill and perseverance prevail at all times. This is all the more remarkable in view of the fact that the plant and equipment, through a series of circumstances, is a hybrid development and adaption. Its present use is far from that for which it was originally designed.

Here is what happened to the plant. The tunnels were originally under contract to the Mason & Walsh Co. This contractor drove 14x16-ft. center pilot headings the full length of each tunnel, which average 6,097 ft. long, as required by the specifications (*ENR*, May 23, 1935, p. 735). Under this first contract the tunnels were intended to be enlarged to an excavated circular diameter of 32 ft. 2 in., with ring-beam ground-supports encased in a 21-in. thickness of plain concrete lining. A second contract was to be let for a lining of steel pipe embedded in 16 in. of reinforced concrete and grout in each tunnel. The finished diameter of the tunnels was to be 26 ft.

About a year ago the contractor started the enlargement work, and a short time later concrete lining got under way, both carried on from the lower portals only (*ENR*, Aug. 29, 1935, p. 296). The full heading method of enlargement was used, blasting from ring-drilled holes carried far ahead of the face from within the pilot headings. The plant designed for the enlarging included a large jumbo, reaching nearly to top of tunnel, equipped with five movable platforms from which the upper reaches were trimmed and ring steel was set. Mucking out was by two mucking machines, working side by side, each loading into a long belt conveyor that extended over a trainload of muck cars. The concrete lining plant included invert forms for gravity placement, and a huge arch form, filled by twin concrete placement machines mounted on cars riding the tunnel service tracks.

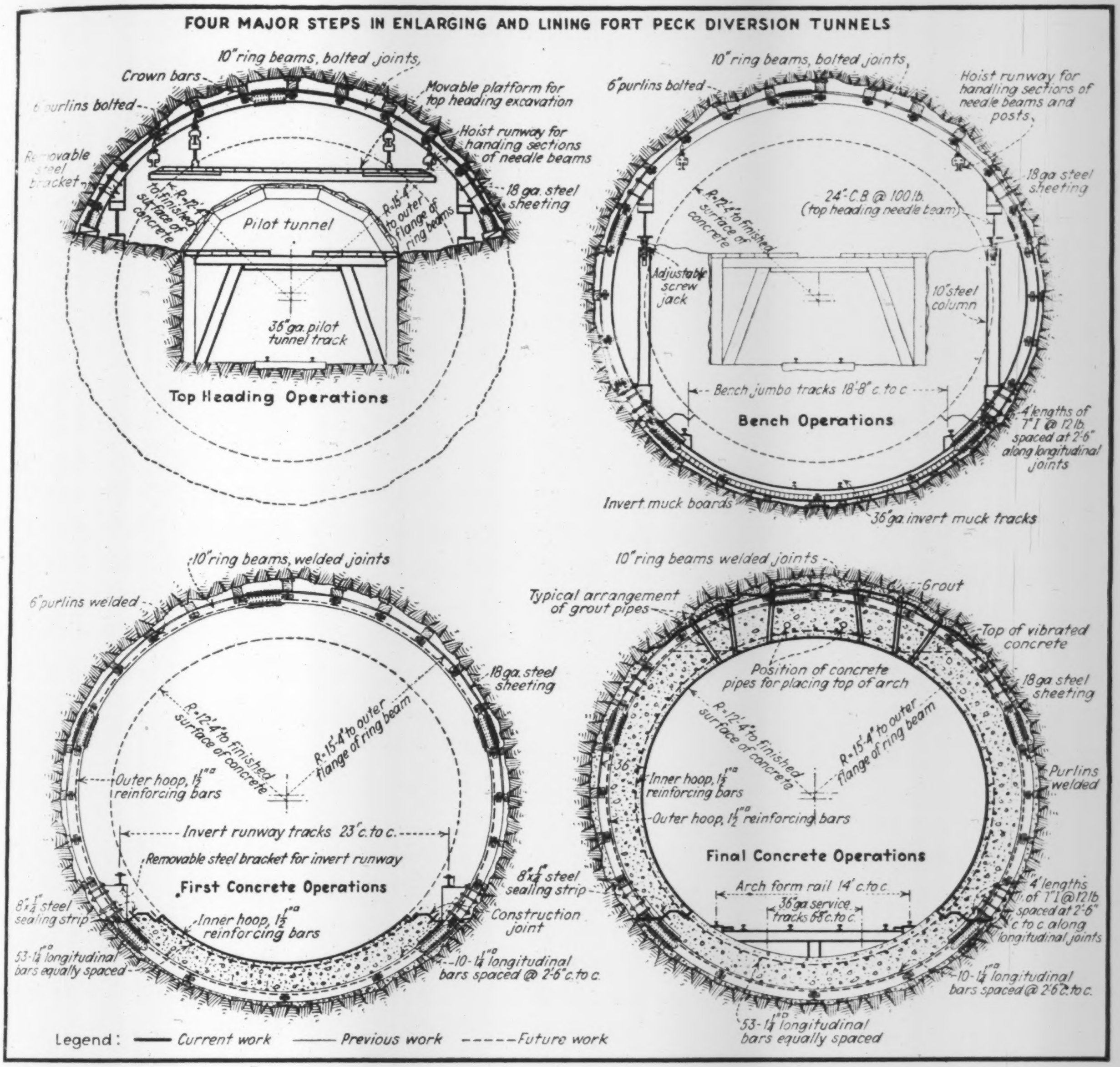
Holding the shale roof by the full heading method proved dangerous and slow. Therefore the top heading and bench method was resorted to in some of the tunnels for a while, but the contractor did not fully develop this method and it was never given a thorough trial.

In January of this year tunneling plans were suddenly changed. The design was altered to an excavated diameter of 30 ft. 8 in., the ring-beam ground-support was retained, but a single 36-in. lining, heavily reinforced, replaced the two-part lining of concrete and steel, bringing the finished diameter down to 24 ft. 8 in. The steel pipe was eliminated, except for the lower part of tunnel No. 1, which is being fitted up for a possible future penstock. At the time of the change, the contract was terminated by mutual agreement, and the government took over all materials, plant and equipment, and most of the working force, with a view to completing the work in a single operation. From then on the tunneling has been carried on by day labor under direct supervision of the army engineers (*ENR*, Mar. 26, 1936, p. 450).

The contractor had a sufficient number of steel ring beams of the larger diameter on hand to carry all four tunnels as far as the shafts. To prevent the delay in operations that would have resulted from waiting for new rings to be fabricated and shipped, and to save the cost of scrapping the old beams the engineers decided to carry the large-size excavation as far as the shafts, but to install a single 45-in. concrete lining so as to bring the finished diameter down to the new size. In those sections already fully lined to the old size an inner lining will be placed, but this will be done at a later date.

Immediately upon taking over the work the army engineers decided to gradually abandon the full heading method in favor of top heading and bench. This required a double change in equipment—first to cut it down to fit the smaller diameter, and second, to adapt it to the revised method of tunneling. Without stopping work the plant was altered as rapidly as possible. Yet it could not all be changed at once, for there was some of the large-size excavation remaining. This change in size and new adaption of the plant wasn't the whole story, however, for those in charge introduced several new and revised operations that required further alterations and additional equipment.

Adoption of the single-lining plan necessitated two additional moves. First was the placement of heavy bar reinforcing, not included in the original lining. Second, as the ring beams were designed as part of the reinforcing, it was necessary to weld all bolted field joints to prevent possible movement under stress changes from empty to full tunnel under high hydrostatic heads. Rings are erected with bolts, but before they are encased with concrete welding crews put a bead around all splice plates, making the rings an integral frame. Purlin connections are also welded to develop them as continuous longitudinal reinforcement.



FOUR MAJOR STEPS IN ENLARGING AND LINING FORT PECK DIVERSION TUNNELS

10" ring beams, bolted joints
Crown bars
6" purlins bolted
Removable steel bracket
Movable platform for top heading excavation
Hoist runway for handing sections of needle beams
18 ga. steel sheeting
R=12'4" to finished surface of concrete
R=15'4" to outer flange of ring beams
Pilot tunnel
36" ga. pilot tunnel track
Top Heading Operations
10" ring beams, bolted joints
6" purlins bolted
Hoist runway for handling sections of needle beams and posts
18 ga steel sheeting
R=12'4" to finished surface of concrete
24"-C.B @ 100 lb. (top heading needle beam)
Adjustable screw jack
10" steel column
Bench jumbo tracks 18'-8" c. to c
Bench Operations
4' lengths of 7" I @ 12 lb. spaced at 2'-6" along longitudinal joints
Invert muck boards
36" ga. invert muck tracks
10" ring beams, welded joints
6" purlins welded
18 ga. steel sheeting
R=12'4" to finished surface of concrete
R=15'4" to outer flange of ring beam
Outer hoop, 1½"ø reinforcing bars
Invert runway tracks 23' c. to c.
Removable steel bracket for invert runway
First Concrete Operations
8"×¼" steel sealing strip
Inner hoop, 1½"ø reinforcing bars
8"×¼" steel sealing strip
Construction joint
53-1¼"ø longitudinal bars equally spaced
10-1¼"ø longitudinal bars spaced @ 2'-6" c. to c.
10" ring beams welded joints
Grout
Typical arrangement of grout pipes
Top of vibrated concrete
Position of concrete pipes for placing top of arch
18 ga. steel sheeting
R=12'4" to finished surface of concrete
R=15'4" to outer flange of ring beam
36"
Inner hoop, 1½"ø reinforcing bars
Outer hoop, 1½"ø reinforcing bars
Purlins welded
Final Concrete Operations
Arch form rail 14' c. to c.
36" ga. service tracks 6'8" c. to c.
4' lengths of 7" I @ 12 lb spaced at 2'-6" c. to c. along longitudinal joints
10-1¼"ø longitudinal bars spaced @ 2'-6" c. to c.
53-1¼"ø longitudinal bars equally spaced
Legend: —— Current work —— Previous work ----- Future work

FIG. 1—DIAGRAMMATIC SKETCH of four major operations in enlarging and lining the tunnels. Welding of the ring beam joints is done just prior to the start of concrete placing; grouting follows placement of the arch lining.

Top heading operations are carried on from the upstream portals, utilizing the pilot tunnels, though work progresses toward the portals. The first move is to erect a working platform within the pilot bore, the deck of which is at the spring line of the small tunnel, and also at the level of the floor of the top heading. This platform is carried on timber posts, canted inward, with heavy cross beams bracing the original lining posts of the pilot tunnel. The whole is designed to support effectively the sides of the small tunnel after the arch timbering of the pilot tunnel has been removed in top heading driving, and further to serve as a floor upon which muck from the top heading is caught when blasting. This timbering is carried 200 or 300 ft. ahead of the bottom bench face.

In the top heading work is the first of many simple but ingenious contrivances that make for rapid progress. This is a traveling platform, rolling on trolley beams hung from the top ring steel, and clearing the steel about 5 ft. at the crown (Fig. 6). From this platform the top part of the heading is drilled and loaded, the roof is scaled and crown bars and ring beams are set. All drilling equipment and other tools are left on the platform during blasting, for it is merely rolled back out of the way.

All drill holes are horizontal, except for a slight turning out of the outside holes and a slight cut at the center. Holes are drilled with jackhammer type drills fitted with auger bits. In very good ground, holes are drilled to a depth of 8 ft., pulling enough ground to allow erection of two sets of ring beam segments (rings are 3½ ft. c. to c.), but in bad ground 4-ft. holes are drilled, giving an advance of only one ring of steel. Because of the varied character of the ground there is no standard drilling and shooting procedure. Sometimes the advance is entirely by chipping. Usually, 30 per cent straight gelatin dynamite is used, and gives an average powder factor of 0.35 lb. per cu.yd. Shooting is in delays up to four in number. Just before shooting, the arch of the pilot tunnel timbering, extending above the floor platform, is removed for the length of the advance.

After shooting, the sides and top are

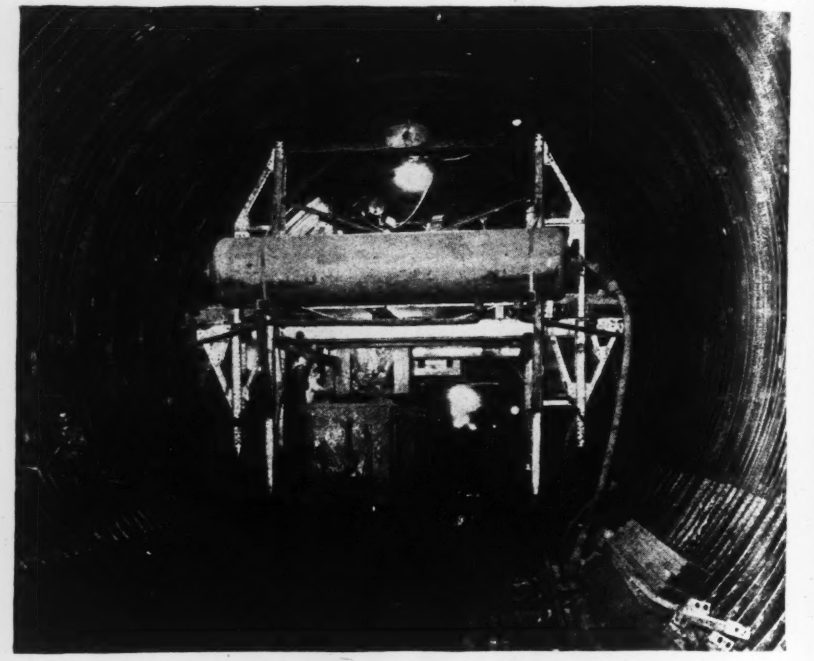
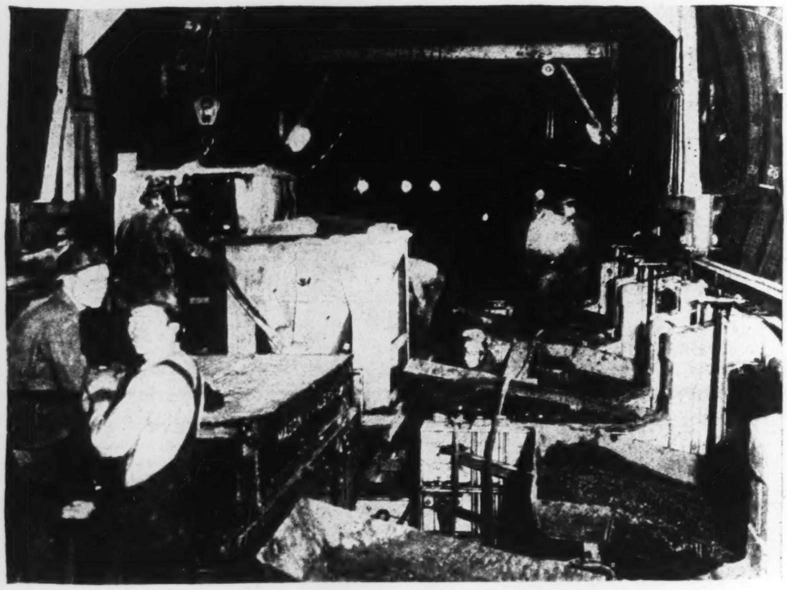


FIG. 2—ARCH-LINING GANTRY from the front, looking towards the arch form. The concrete guns are located outside the gantry legs, at the lower corners, clearing the service railroad tracks. A bucket of concrete is being hoisted up the clear well.

FIG. 3—INVERT CONCRETE is poured from concrete buckets hoisted over sidepans discharging into the form.



barred down by hand, trimmed with air chisels, and given the first of three coats of sealing solution. Then crown bars, of 6x6-in. to 10x10-in. timbers of various lengths, are inserted if needed to block up the roof. The bars are set behind the last-erected ring steel, bearing on the steel and blocked against the roof behind the steel as a cantilever support for ground just exposed.

Next, the steel ring support is set, in two segments of the full circle jointed at the crown. The ring steel is 10-in. H section. Adjacent rings are tied together with 6-in. I-beam purlins. Rings are fitted with temporary shoes and blocked at their lower ends, resting on the solid shale floor of the top heading.

As the shale is dead, flyrock is scarce, and the muck piles up close to the face. Handling of muck is another simple, ingenious process. The center panel of the working floor is removed, below which is spotted a string of muck cars. Then the muck is scraped through the center opening into the cars by means of a power hoe handled by a double-drum air hoist, as shown in Fig. 7. Experience has taught how and where to make the hitches for the pull and back lines, and the operators have become expert in maneuvering the hoe. Flaring wings on the scraper clean out the corners.

Two more coats of sealing solution are applied to the exposed shale as work progresses. To protect the workmen from falling chunks of shale that might work loose later, two strips of 18-ga. steel are worked around the back sides of the purlins in each bay. These sheets form a substantial protection.

When driving the pilot heading in No. 1 tunnel a section was encountered that was badly faulted and carrying some water. To get this section out of the way before regular top heading operations reached it, a raise was made from the pilot bore from which top headings were carried both ways. In the worst part, the ring segments rested on 24-in. steel beam wall plates. In side drifts carried from the small tunnel, steel posts were set to support these wall plates. In addition, to give continuous support to the roof over its whole area, steel plates were set between the rings and low pressure grout was forced between ground and plates.

Bottom headings

The high jumbo designed for full-face heading operations has been cut down to fit the bottom heading, and travels on rails set on brackets on the lower steel ring segment. From its moving platforms, drilling, trimming, and like work are carried on about the same as at the top heading. All holes are drilled horizontally, in a pattern to suit the ground, and are loaded with varying charges of powder. A mechanical mucker loads out the muck, dumping into one of the original belt conveyors discharging into a train of muck cars. All bottom heading work is carried on from the lower portals. Wooden lagging placed in the invert catches the muck and makes its removal easy, and minimizes the quantity to be cleaned out by hand ahead of concrete placing.

An ingenious temporary wall plate or needle beam is used for carrying the load of the top segments of ring beams at the bench face (Fig. 8). Along each side of the top heading, with plenty of bearing on solid ground, is set a line of **24-in., 100-lb. H beams.** The individual sections of the beams are 10 ft. long, seven sections being fastened together end to end by full-strength bolted splices. Near the lower ends of each top segment of ring steel is fastened a temporary bracket, bearing against a short bar welded to the inside face of the ring steel.

In the top heading section the beams are blocked up against the brackets by steel wedges. The beams extend for 10 to 20 ft. back into the area of completed rings in the bottom heading. Here, the

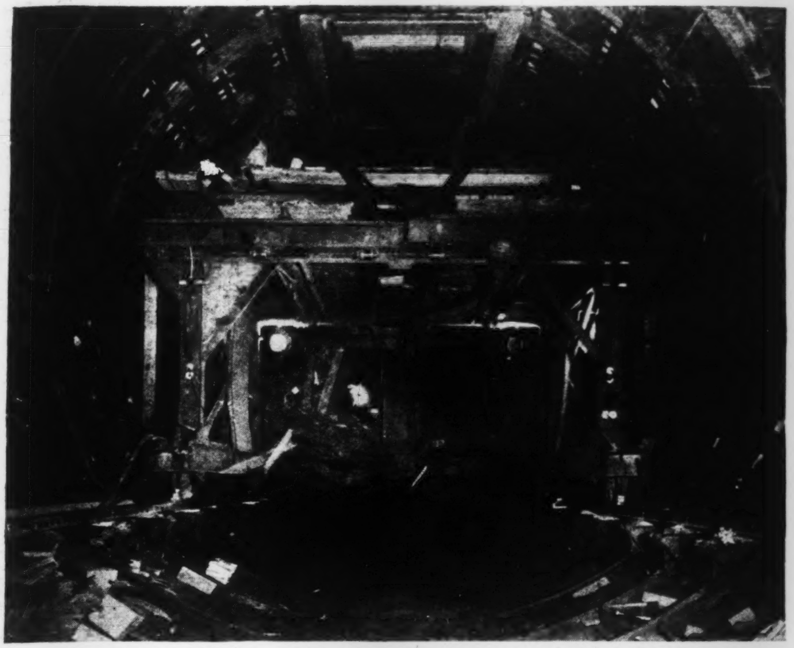
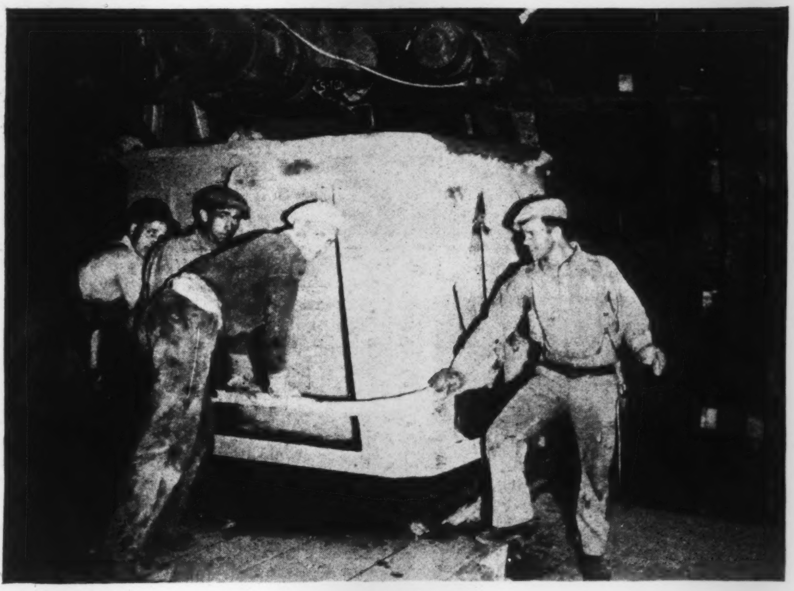


FIG. 4—A JUMBO with a sliding top platform aids in bench excavation and setting of lower segments of the ring beams. Steel posts carrying the rear end of the needle beam can be seen outside of the gantry legs.

FIG. 5—CONCRETE GUNS on the arch lining gantry are charged from overhead by hoisting the buckets up a clear well in the gantry frame. The trolley carrying the hoist curves from the clear well at the center to a position over either gun.



free end of the beams are carried by a series of steel posts, capped with screw jacks and resting on brackets on each ring similar to those bearing on the top of the beams. The center section of the beams is unsupported; here bench excavation is carried on and the lower segment of rings is erected. Thus, the top steel is supported over the transition between top heading and full section. As work progresses, the rear beam sections are taken out and moved ahead to the front, far up in the top heading.

Two chain hoists traveling on a trolley beam hanging from the top steel handle the beam sections, as well as the posts. The temporary wall plate across the transition zone works well and holds the roof undisturbed until the load is picked up by the full rings.

The welding crew follows close behind the bench excavation. Six welding machines mounted high on a gantry that clears all trains and provides working platforms are used for welding of the rings and purlins. And following close to the welders is the reinforcing gang that places all of the outside course of bars and those on the inside for the invert. All reinforcing is bent on the job by a power bending machine. Bent bars are carried into the tunnels in special well-cars in the position they are to be used in the tunnel—that is, invert bars are loaded with curved ends upward, arch bars with curved ends downward.

Concrete lining

The first operation in placing the concrete lining is to pour the bottom quarter of the tunnel as an invert. A 70-ft. form, with the bottom open for screed finishing, is hung from gantry rails resting on brackets high on the ring steel. Concrete is brought into the tunnel in 2-yd. bottom dump buckets on flat cars. The buckets are lifted from the cars by an overhead traveling hoist and are dumped into shallow side chutes placed at the top of the form, as shown in Fig. 3.

From the lower portals up to the invert form, the service railroad is double tracked. Through the invert and on to the bench excavation, there is only one track. To accommodate the change in grade between top of invert form and bottom of tunnel beyond the form, a movable ramp of structural steel is used.

Combination trolley and battery mine-type locomotives haul all muck, concrete and steel trains within the tunnels. In the completed tunnel, trolley wires supply the current for operation of locomotives, but beyond the arch form batteries are required. The advantage of this dual power supply is that whenever the motors are standing idle in contact with the trolley wire, the batteries are charging. This minimizes the time spent in the charging station.

The plant for placing sidewall and arch concrete is a marvel. A 70-ft. length of steel form covers the three-fourths of the tunnel circumference above the invert. Along each side of the form are two rows of five windows each, the top row is just above the springline. Through the top row the sidewall concrete is placed; the lower row is used for access of workmen to the inside of the form and for inspection. As long as men can work inside of the form in the rising concrete, the mix is vibrated with pneumatic internal vibrators.

Twin ¾-yd. placing machines are mounted on the lower sides of a separate traveling gantry, clear of the two service tracks (Fig. 2). Immediately above each gun is a 2-yd. charging hopper. High on the gantry, close to the top of arch, is a horseshoe-shaped hoist track, curving over a clear well in the gantry frame to position over either charging hopper. Two electric traveling hoists are mounted on the curved track. Concrete is brought to the gantry in trains of four 2-yd. buckets each. The overhead hoists lift the buckets, one at a time, to a level high enough to swing over the charging hoppers, where

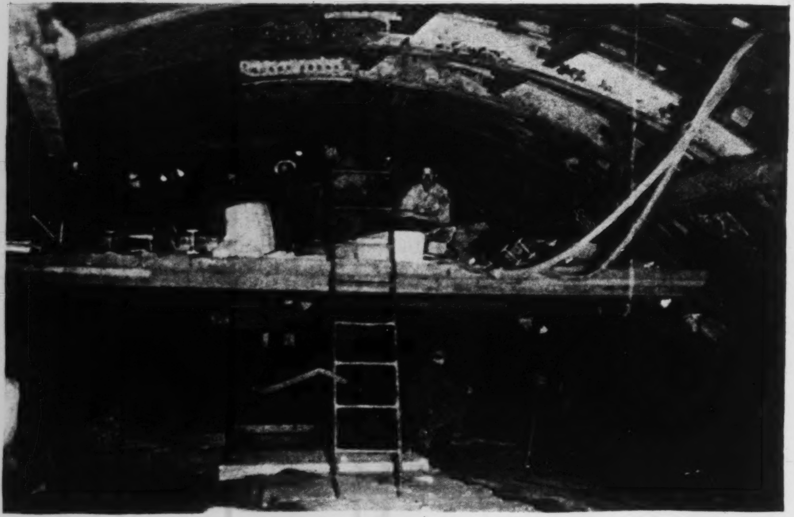


FIG. 6—TRAVELING PLATFORM hanging from top ring beams is one of several ingenious devices that are speeding up driving of Fort Peck diversion tunnels.

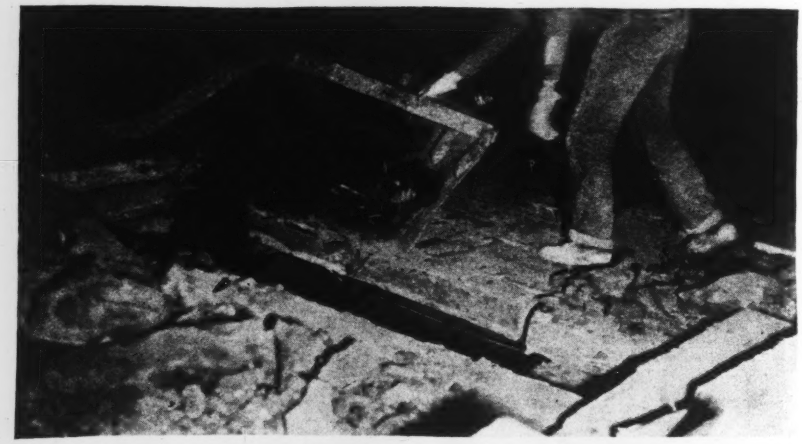


FIG. 7—POWER SCRAPER sweeps top heading muck into waiting cars beneath the working floor set up in the pilot tunnel.

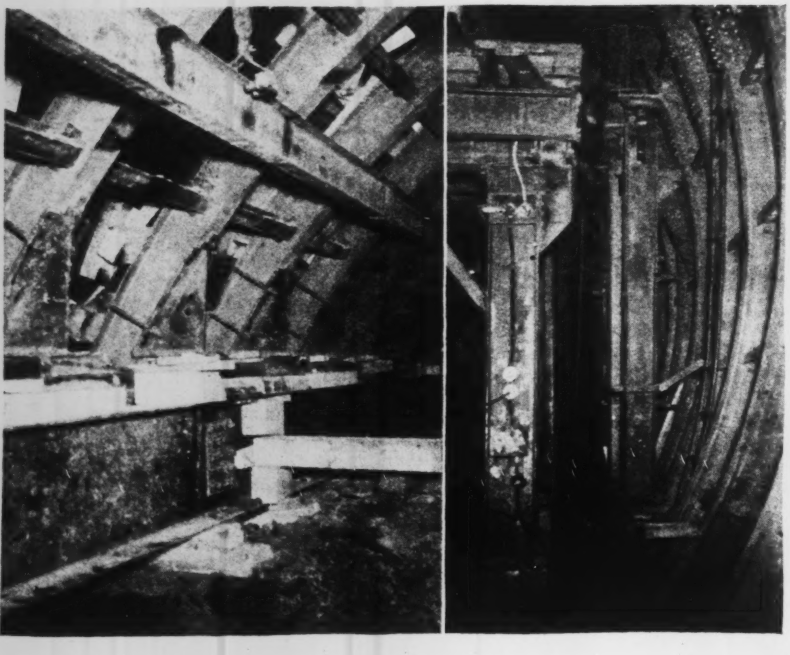


FIG. 8—SECTIONAL NEEDLE BEAM, blocked up on top heading floor at front and carried on temporary posts at rear, supports roof beams over the transition from top heading to full section excavation. Left view is that of the beam at bench face, with the chain hoist trolley beam overhead; the right view shows the posts carrying the rear end of the needle beam, set on brackets fastened to the completed rings. The structure at the extreme left of this view is the jumbo used in steel erection.

they are dumped in an instant. To save a moment's time, an empty flat is carried in each train and the last empty bucket remains at the gantry until the next train arrives.

From each gun lead two discharge lines, one going into the opening under the form, at the level of the top row of windows on each side; the other extending into the arch. A supporting gantry between the gun gantry and the form helps to carry the four long discharge pipes. A rubber hose connects the discharge end of the gun with either of the two lines on that side, as desired.

At the start of a pour, the guns discharge through the lower pipes. Short side chutes conduct the concrete from end of discharge line through the form openings into the sidewalls. It is the practice to start at the far end and fill the form for a depth of 3 ft. or so, then, by means of quick-acting connections, shorten the pipe to permit discharging into the next form opening. Concrete filling continues on both sides of the form at once, the pipes discharging through successive openings to extend the 3-ft. lift. When the end of the form is reached, the process is reversed, and the pipe is successively extended, filling towards the rear for an additional 3 ft. of lift.

After the sidewalls are filled in successive lifts to the level of the form openings, the gun leads are changed to the arch pipelines, which enter the form through the end bulkhead at the crown. These pipes are pulled back and forth in the form by moving the entire assembly of pipes and gun gantry by means of a locomotive. For a while the pipes discharge through side chutes that travel with the pipes. The pipes are moved back and forth to give a uniform rise of concrete in low lifts. As the arch fills up, the side chutes are taken out and finally concrete discharges direct from the pipes.

The endwise movement of the top pipes with their side chutes naturally requires a clear space. To provide this space, inside reinforcing bars are blocked up on the form by precast concrete blocks; outside bars are tied to the ring steel. Grout pipes are inserted in the form, and to clear the chutes requires a novel arrangement of these pipes. A short nipple is tack welded to the ring beams, tapped with a set screw. The grout pipes are inserted inside of these nipples, pushed upwards into the clear, and are held there temporarily by tightening up the set screws. After the side chutes have been removed, the grout pipes are released, dropped down to position against the form, though still protruding from the nipples at the upper end, and are wedged tight.

Incidentally, the upper part of the inside course of reinforcing bars is set from the intermediate gantry prior to moving the form ahead.

At present, an arch form is filled in one tunnel while an invert section is being poured in another tunnel. This gives one complete 70-ft. section of lining per day, or one in each tunnel every four days. All tunnel work continues night and day. An arch pour requires from 16 to 22 hr.

Behind the concreting comes the grouting of the crown, which will be described in the next article in this series.

All of the above operations are carried on independently in each of the four tunnels. Each tunnel has its own complete inside plant, identical with the others.

At this time, lining of the four 50-ft. control shafts and the four 16x34-ft. emergency gate shafts, all about 250 ft. deep, is just getting under way. Concrete is transported in buckets by trucks to top of shafts. Guy derricks lower the buckets to a working platform within the smaller shafts, from where concrete is spouted into the rising forms.

For the big shafts, concrete is transferred to a special 4-yd. homemade bucket, equipped with a ball-controlled bottom opening. This bucket is lowered by the derricks directly over the form to be filled. Steel forms are used in all shafts. A steel interliner complicates the lining work, for the steel must be welded in place.

Lack of space prohibits a complete description of the outside plant, interesting as it may be. At both the upper and lower portals are elaborate central concrete plants, charged by belt conveyors from tunnels under aggregate storage piles. The lower portal plant supplies all concrete for the tunnel lining. The upper portal plant mixes for the upper headwall structure, still under construction, and for shaft lining. At each portal tunnel muck is disposed of by 2,500-ft. belt conveyors.

Direction

The tunnels are being built by the day labor method under the general direction of Lt.-Col. Thos. B. Larkin, district engineer. Maj. Clark Kittrell is chief of operations, and Capt. A. W. Pence is in direct charge of all tunnel work. O. F. Brinton is construction superintendent.

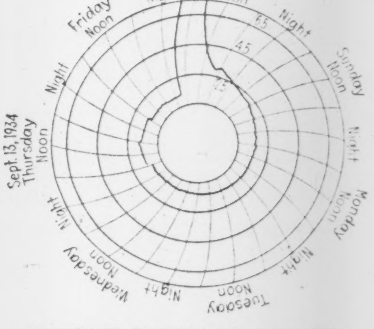
Rainfall and Runoff in Colorado

Unusual rainfall intensities, and corresponding runoffs from drainage basins in eastern Colorado, within the last three years, have been so greatly in excess of anything of record, that special investigations have been undertaken, to accumulate data upon which to base the designs for future proposed storage reservoirs in those and similarly affected areas.

The areas covered by these studies, herein reported, consist of the Kiowa and Bijou, forming parts of the drainage basin of the South Platte River, and of the Republican and Arickaree rivers in northeastern Colorado, and the Fountain, Horse and Granada creeks, tributaries of the Arkansas River,—all located east of the Rocky Mountains. Editor.

By **M. C. Hinderlider**
State Engineer of Colorado, Denver

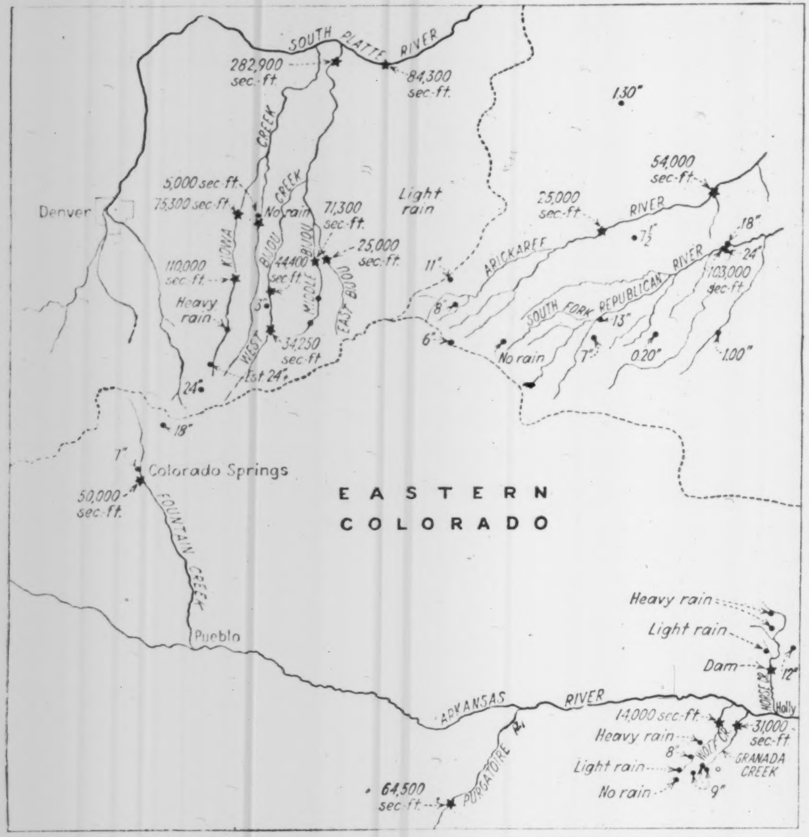
For many years the engineering profession has realized the need for greater knowledge of intensities of rainfall and resultant runoff from well-defined areas for the purpose of arriving at more rational conclusions in connection with the design of flood-control reservoirs, spillway capacities, highway and railroad bridge clearances, channel rectifications and other flood-control works. The most important factor in the design of spillways for reservoirs, bridge clearances and channel rectification works is the maximum momentary peak flow in second-feet. With respect to flood-control reservoirs, the most important factor is the total volume of runoff that may be expected to result from the drainage basin above the reservoir in question. The damages usually resulting from flood discharges along a natural water course are caused by the momentary peak flows rather than by total volume of flow.



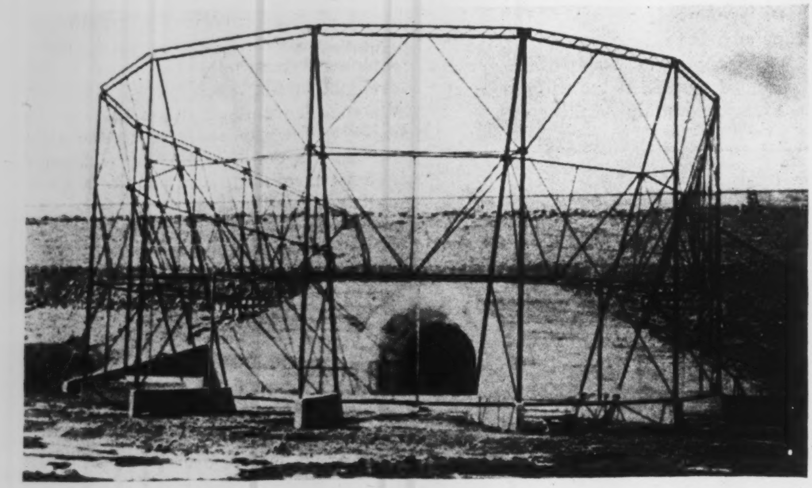
RECORDING CHART for Purgatoire River at Nine Mile dam showing the gage heights for the 7-day period beginning Sept. 13, 1934.

Increasing rates and volumes of runoff from drainage areas throughout the state, which are very largely the result of devastation of vegetable cover, the plowing up of precipitous slopes and unusual combinations tending to intensify high rates of runoff, call for planning of flood-control works of various kinds. Such works, unless they involve the construction of flood-control reservoirs, do not necessarily create a menace to life and property along a water course. However, where such flood-control devices involve the creation of storage reservoirs, particularly above centers of dense population or highly developed areas, such works constitute in some degree a menace to life and property downstream. In consequence, prior to the creation of such a menace, the most careful thought should be given to all factors affecting the safety of such dams, one of the most important requirements of which is that the dam shall be of such height that there will be no possibility of its being overtopped from lack of sufficient storage capacity. The importance of all possible data, based upon authentic records of runoff, coupled with rational deductions therefrom, must, therefore, be obvious.

The phenomenal rainfall over, and runoff from, many of the drainage basins in the easterly part of Colorado in 1933, 1934 and 1935 raises the question as to what, if any, influences or effects the equally phenomenal meteorological conditions, which occurred during a portion of this period, had upon such rainfall and runoff. Some of the highest temperatures ever recorded in Colorado occurred in the summer of 1933. The great drouth occurred in 1934 and, following this drouth, there occurred the great dust storms in the spring of 1935, which were followed almost immediately by the phenomenal rainfalls and runoffs in question. It is the general belief that conditions which control air


SOUTH PLATTE RIVER
282,900 sec.-ft.
84,300 sec.-ft.
1.30"
KIOWA CREEK
BIJOU CREEK
WEST BIJOU
MIDDLE BIJOU
EAST BIJOU
5,000 sec.-ft.
75,300 sec.-ft.
No rain
71,300 sec.-ft.
Light rain
25,000 sec.-ft.
Denver
110,000 sec.-ft.
44400 sec.-ft.
3"
Heavy rain
34,250 sec.-ft.
Est. 24"
24"
18"
54,000 sec.-ft.
25,000 sec.-ft.
ARICKAREE RIVER
7½"
18"
24"
103,000 sec.-ft.
SOUTH FORK REPUBLICAN RIVER
11"
8"
6"
13"
No rain
7"
0.20"
1.00"
7"
Colorado Springs
50,000 sec.-ft.
FOUNTAIN CREEK
EASTERN COLORADO
Pueblo
ARKANSAS RIVER
Heavy rain
Light rain
Dam
12"
HORSE CR.
Holly
14,000 sec.-ft.
31,000 sec.-ft.
Heavy rain
8"
WOLF CR.
GRANADA CREEK
Light rain
No rain
9"
64,500 sec.-ft.
PURGATOIRE R.

RAINFALL AND RUNOFF during heavy storms of 1935 in Eastern Colorado.



DISCHARGE CONDUIT under Kenwood Dam. The steel frame supports a net to catch driftwood.

currents and precipitation are almost wholly the result of differences in temperature and barometric pressures. It is thought that the wind storms were probably no greater than those which have prevailed, in former years, but were impressed upon the minds of the people because of the great amount of dust which they carried. The latter condition is the result of the destruction of the grass and much of the vegetable cover of the Plains areas, caused by the preceding year's drouth, overgrazing and the plowing up of the native sod.

The impression generally prevails among irrigationists and residents of the Plains area, that the great dust storms in the spring of 1935 tended to seal the surface pores of the water courses and their drainage areas, thereby reducing their absorptive capacity, and hence increasing the rate and amount of runoff from such areas. Studies also disclosed that many of the drainage areas had become partially or thoroughly saturated prior to the great floods in question as a result of fairly continuous precipitation covering periods of from a few days to a month in time. This latter condition prevailed over the drainage basin of Kiowa and Bijou creeks.

It will be apparent that the phenomenal rates and volumes of runoff from many of the drainage areas in the Plains region of eastern Colorado in the summer of 1935 were the result of a combinaiton of conditions particularly conducive to such high rates of runoff, these being: First, the destruction of much of the grass sod overlying the drainage basins by the great drought of 1934; second, the cementing of the porous surface soils as a result of the great dust storms; third, the intensity of rainfall over this area as a result of barometric conditions; fourth, in certain areas, the previous saturation of the drainage areas by copious and, in some instances, almost continuous precipitation. This combination may never occur again, or may occur again within a few years' time. But the probability of such recurrences, we take it, is rather remote. Under such a combination of conditions occurring over any drainage basin of an area of several hundred square miles or less, any attempt to control the runoff resulting from storms of such magnitude, through the construction of reservoirs, must of necessity invite grave hazards to such structures and to life and property downstream. In many areas in Colorado east of the Rocky Mountains a great need exists for flood-control projects. Many requests are made of the state engineer to investigate and report upon their feasibility. In many instances where such investigations or studies have been made by us the indications are that the construction of such projects would have the effect of creating a menace rather than providing a safeguard.

Purgatoire Flood—The flood of Sept. 15, 1934, on the Purgatoire River occurred over the middle section of its drainage basin. A satisfactory record of the rate of flow, the peak flow and the total runoff in acre-feet was obtained at the Nine Mile dam, from the automatic register chart. The total drainage area above Nine Mile dam amounts to 1,008 sq.mi., but this rain covered 635 sq.mi. only. The precipitation varied from 1 to 5 in. in depth and was of comparatively short duration, but the rainfall was so intense in certain parts of the area as to produce a runoff in the form of sheet water which destroyed sections of the highways and railroad.

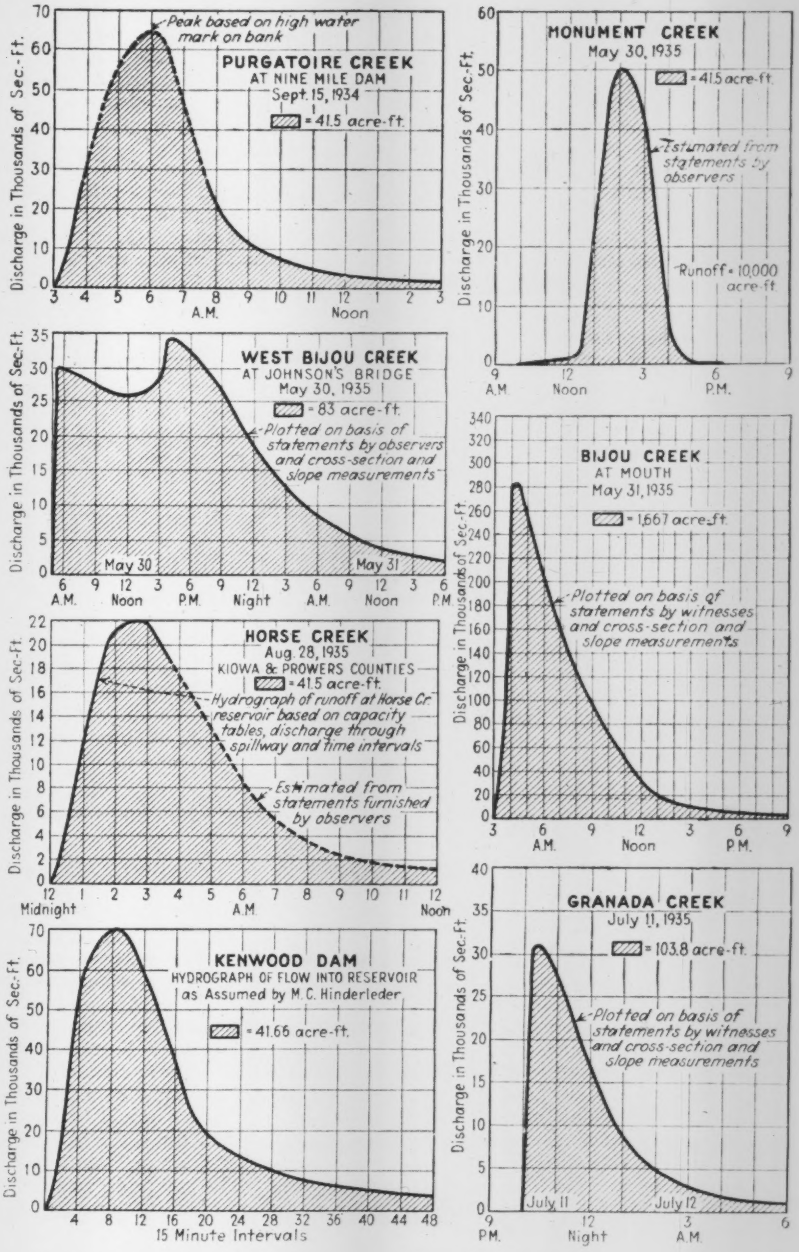
The recorded peak flow over Nine Mile dam, which is a low concrete structure, was 64,200 sec.-ft. or 64 sec.-ft. per sq.mi. for the entire drainage basin above the dam, and 101 sec.-ft. per sq.mi. from the drainage area over which the rainfall occurred. The total runoff amounted to 22,800 acre-ft. or 36 acre-ft. per sq.mi. for the 635 sq.mi.

Horse Creek Flood—As is characteristic of the drainage area of the Purgatoire basin, the drainage area of Horse Creek is almost devoid of vegetation, except the natural prairie sod, a portion of which has been plowed up. The upper part of the drainage basin consists of gently sloping areas containing a number of dry lake beds which tend to retard runoff. The middle sections of the drainage area are more precipitous, while the lower section of the basin increases in slope and is traversed by numerous tortuous and fairly deep arroyos.

The flood of Aug. 28, 1935, resulted in the destruction of the Horse Creek flood-control reservoir dam, located some 9 or 10 miles above Holly. (See *ENR*, Sept. 5, 1935, p. 341.) This dam had just been completed by government relief forces. The dam had a maximum height of 40 ft., length of 2,400 ft., reservoir capacity at spillway elevation of 2,675 acre-ft., and 3,650 acre-ft. at crest of dam. A 225-ft. spillway around the westerly end had a depth varying from 5 to 7 ft. With the water at the top of the dam, the spillway had an estimated capacity of 16,000 sec.-ft.

A fairly reliable record of the volume and rate of runoff from the drainage area above the dam was made possible from a knowledge of the time intervals at which the water rose in the reservoir and from the capacity of the reservoir and spillway. From such information and also from cross section and slope levels taken on the channel of the creek above the reservoir, it is concluded that the peak flow resulting from this flood amounted to 22,000 sec.-ft., or 142 sec.-ft. per sq.mi. for the total drainage area of 155 sq.mi.

Investigation disclosed that the rainfall on the night of Aug. 27 and 28 ranged from 2½ to 11½ in. The rainfall covered a period of 2 to 4 hr. The area over which the rainfall occurred was about 100 sq.mi. The peak flow would therefore, amount to 220 sec.-ft. per sq.mi. The area over which the rainfall of greatest intensity occurred, approximated 50 sq.mi., which would show a peak runoff of 440 sec.-ft. per sq.mi. The total runoff of the area over which the rainfall occurred would approximate 100 acre-ft. per squire mile. (See Graph.)



HYDROGRAPHS of floodflows in Purgatoire River and in Monument, Granada, Horse, Bijou and West Bijou creeks, also an assumed hydrograph for the flow into Kenmore reservoir under the conditions encountered.

Granada Flood—The 2 to 9-in. rain storm which caused the Granada Creek flood commenced about 7 p.m. July 11, 1935, and reached its greatest intensity about 8:15 p.m., and ended about 3 a.m. July 12. The precipitation over the basin ranged from 2 to 8 in. The determination of runoff was made by the usual slope and cross-section method. This flood is the greatest of which there is any record in this basin and resulted in the loss of eight lives and great losses to livestock and other property.

Colorado Springs Flood—The flood at Colorado Springs on the Monument Creek, a tributary of the Fountain River, originated 2 mi. northwest of Colorado Springs about 10:30 a.m., May 30, 1935, and lasted from 2 to 2½ hr. The creek reached flood stage about 12:30 p.m., crest elevation about 2 p.m., and had receded somewhat by 3:30 p.m. The peak discharge, as determined by F. O. Ray, city engineer of Colorado Springs, was 50,000 sec.-ft. This flood on Monument Creek is the greatest of which there is any record, and created damages to property in Colorado Springs estimated at $750,000 and the loss of three lives.

Cherry Creek Flood—The drainage area of Cherry Creek above Castlewood dam consists of 175 sq.mi. of rather pronounced topography, intersected by several deep and well-defined water courses culminating immediately above the Castlewood reservoir. The upper rim of the drainage basin is heavily timbered with pine and other similar growths, while the lower part of the basin above the dam consists of grass covered and cultivated areas. Following the failure of the dam about midnight of Aug. 2-3, 1933, investigations made by the state engineer indicated that the intensity of the rainfall ranged from nothing to 7½ or 8 in. in depth. The rainfall originated in the upper reaches of the drainage basin in the late afternoon or evening, causing the peak flow in question at the reservoir about midnight. The peak flow measured by means of cross sections and slope showed a discharge of 31,960 sec.-ft. The flood resulted in an estimated total runoff at Castlewood reservoir of 2,620 acre-ft.

Hydrographic data of heavy floods in Eastern Colorado During the last three years

Location	Peak Q Sec.-Ft.	Watershed Covered by Rain: Area Sq. Mi.	Watershed Covered by Rain: Peak Q Sec.-Ft. per Sq. Mi.	Watershed Covered by Rain: Run-off Acre Ft. per Sq. Mi.	Area of Most Intense Rain: Area Sq. Mi.	Area of Most Intense Rain: Peak Q Sec.-Ft. per Sq. Mi.	Area of Most Intense Rain: Run-off Acre Ft. per Sq. Mi.	Est. Total Runoff Acre-Ft.	Data Collected by
Monument Creek	*50,000	130	385	77	108	463	93	10,000	L. T. Burgess
Wolf Creek—above Granada	14,000	75	187	39	25	560	116	2,900	W. T. Ferguson
Granada Creek—above Granada	31,000	40	775	160	35	886	183	6,400	W. T. Ferguson
Castlewood Dam	32,000	131	244	20				2,620	J. E. Whitten
Kenwood Dam (Prelim. Design)	45,000	375	120						F. C. Carstarphen
Kenwood Dam (Final Design)	70,000	375	187	68				25,000	M. C. Hinderlider
Kiowa Creek, 12 mi. north of Kiowa	110,000	190	578	591	120	917	937	112,400	J. E. Whitten & Frances Hart
Kiowa Creek at Bennett	75,300	266	284					75,300	Whitten & Hart
Kiowa Creek at Elbert	43,500	60	725						R. J. Tipton
W. Bijou Creek—Johnson's Brg	34,250	118	291	462	108	317	505	54,500	Whitten & Hart
W. Bijou Creek—18 mi. S. Strasburg	44,400	187	238		172	258			Whitten & Hart
Middle Bijou Creek at Peoria	143,640	230	623						Whitten & Hart
Middle Bijou Creek below mouth of Wilson Ck	71,270	151	473		146	488			Whitten & Hart
West Bijou Creek at Byers	164,670	280	588						Whitten & Hart
East Bijou Creek at Deer Trail	25,000	294	85		38	651			Whitten & Hart
Bijou Creek at mouth, above Bijou canal crossing	282,900	1,444	196	81	387	731	302	116,800	Whitten & Hart

*Computed by F. O. Ray, city engineer, Colorado Springs.

When the flood entered the reservoir, there was already stored therein 2,272 acre-ft. The flood quickly filled the reservoir and overtopped the dam the entire 600-ft. length to a depth of 1⅓ ft. The spillway over the center of the dam had a depth of 4 ft. and a length of 100 ft. The flood was limited almost entirely to the area above the dam. This flood, together with the failure of the dam, created a peak flow at the present Kenwood dam, some 25 miles below Castlewood dam, estimated to have been 34,000 sec.-ft.

Assuming that the area covered by the rainfall covered three-fourths of the basin above Castlewood dam the peak runoff per sq.mi. would amount to 244 sec.-ft. and 20 acre-ft. per sq.mi., or 180 sec.-ft. per sq.mi. for the total drainage area above Castlewood Dam, and but 15 acre-ft. per sq.mi. for the same area.

Following his field studies of runoff from smaller drainage areas within the Cherry Creek drainage basin above Denver, ranging from about 15 to 24 sq.mi., Dan McQuaid reached the conclusion that peak flows from these smaller areas amounted to as much as from 366 to 734 sec.-ft. per sq.mi.

Kenwood Dam Assumptions—Following the great flood of 1913, the Cherry Creek Flood Commission, created to report upon plans for controlling future floods, concluded that a peak flood of 60,000 sec.-ft. was possible.

In his designs for the Kenwood Dam and reservoir, Fred C. Carstarphen assumed a possible peak discharge in Cherry Creek at Kenwood Dam of 45,000 sec.-ft., and concluded that a reservoir with a capacity of 8,500 acre-ft. would be ample.

Following a review of all available data, we concluded that a peak flow of 70,000 sec.-ft. was possible, and adopted the graph given on p. 245 showing the theoretical runoff in acre-feet following a storm of the latter magnitude, and based upon this graph, we concluded that a reservoir of a capacity of 10,000 acre-ft. at spillway level, was required.

This graph shows that the Kenwood Reservoir is capable of regulating a total runoff from the 375 sq.mi. drainage basin above the reservoir, of 25,600 acre-ft., with a minimum resulting freeboard of 4 ft., which would be equivalent to nearly ten times the volume of the flood which destroyed the Castlewood Dam. The reservoir would take care of a flood of 32,000 acre-ft. based on the accompanying graph or more than twelve times the volume of the flood of 1933, before the dam would be overtopped. The equalizing effect of a 25,600-acre-ft. flood would take care of a runoff of 68 acre-ft. per sq.mi., or the equivalent of the runoff per square mile produced by the 1921 Pueblo flood, and 1.9 times that per square mile of the flood which occurred on the Purgatoire River in September, 1934. With the water surface 4 ft. below the top of the parapet wall, the maximum discharge below the dam will be 26,000 sec.-ft. The estimated 34,000-sec.-ft. peak discharge at the Kenwood dam site on Aug. 3, 1933, when the Castlewood dam failed, caused a peak flow through the Cherry Creek retaining walls in Denver of 16,500 sec.-ft. On this basis a discharge through the conduit and over the spillway of the Kenwood dam of 26,000 sec.-ft. should produce a flood at Denver of about 12,000 sec.-ft. For a peak flow into the reservoir of 60,000 sec.-ft. based upon the type of graph herein shown it is anticipated that very little water would be discharged over the spillway. The discharge through the conduit would not exceed 9,000 sec.-ft. which in turn would create but a nominal flood through Denver.

Arickaree and South Fork Floods—The heavy rain storms originating in the Colorado Springs area, traveling north and eastward along the northern edge of the divide between the Arkansas and South Platte drainage basins caused excessive floods in the basins of the Arickaree River and South Fork of the Republican River. Rainfall data over these watersheds was collected by Robert Follansbee, district engineer of the U. S. G. S. and by L. T. Burgess. A 2-hr. rainfall of 9 in. was reported to have occurred near Seibert during the late afternon of May 30. Near Idalia, on the South Fork of the Republican, more than 24 in. of rain, measured in a stock tank, fell from 6 p.m. to shortly after midnight. Rainfall from 8 to 14 in. was measured and recorded at many points in the two drainage areas.

During and after these excessive rains, both the Arickaree and Republican rivers overflowed their banks and sent enormous volumes of water down these streams. At the height of the flood, skies over the eastern counties were a dark brown due to a most unusual situation. Along the Colorado-Kansas border there was a heavy dust storm. Clouds of dust could be seen for miles and the coppery-hued sky cast a brown shadow, giving the scene a weird appearance.

From the rainfall data collected the drainage area covered by the storm has been outlined. The maximum discharge has been computed by cross-section and slope measurements, which were made by the engineers of the U. S. G. S. On the South Fork of the Republican a cross section was taken south of Idalia near Newton, Col., 10 miles east of the Colorado-Kansas line. The total drainage area above the section covers 1,270 sq.mi. The total maximum discharge was computed at 103,000 sec.-ft. The runoff per square mile from this area is 81 sec.-ft. At Benkleman, Neb., the maximum discharge computed totals 150,000 sec.-ft. from 2,580 sq.mi., which shows a runoff of 58 sec.-ft. per sq.mi.

In the area of 669 sq.mi., drained by the South Fork, a section of intense precipitation was segregated between Newton and north of Seibert. A computed discharge of 83,000 sec.-ft. gives a runoff of 124 sec.-ft. per sq.mi. This is the largest unit runoff computed for the South Fork drainage.

Along the Arickaree River, several miles north, was found excessive rainfall, with attendant extreme floods. At

Cope, Colo., the flood waters descended in two separate peaks. The maximum computed at Cope was 25,000 sec.-ft. from a drainage area of 690 sq.mi. Nine miles north of Idalia, Colo., a slope measurement was made on the Arickaree River. The maximum discharge was computed to be 54,000 sec.-ft. From a drainage area of 1,260 sq. mi. the unit runoff was 43 sec.-ft. per sq.mi.

All old-time settlers claim these floods to be the largest of the last 60 years. Several red cedar roots and stumps were uncovered and several skulls larger than those of cattle or buffalo were also washed up. No red cedar trees now exist in the Arickaree drainage basin and no record of any having grown there was obtainable.

Bijou and Kiowa Floods—The floodflows of May 30, 1935, over the drainage area of Bijou and Kiowa creeks and over the head waters of the Arickaree and South Fork of the Republican appear to have been the greatest by far, of which there is any record or legend. Following these floods assistants of the state engineer's office made several investigations, resulting in the following tentative conclusions, which appear incredible in certain instances (see table). The peak flows in second-feet were ascertained by the usual method of cross-sections and slope levels taken at different points in the several drainage basins. Reliable information could not be obtained relative to the probable runoff in acre-feet, for lack of authentic information concerning the exact period of time covered by the floods in question.

The information obtained of the flood on West Bijou Creek at Johnson's bridge justifies, in a measure, the assumption shown by the accompanying graph which has two peaks, different in shape and form from the graphs of the Purgatoire, Horse Creek and Cherry Creek floods. The graph of the flood at the Johnson bridge shows a peak discharge of 34,250 sec.-ft. and a total probable runoff of 54,500 acre-ft. from a drainage area of 118 sq.mi., or 462 acre-ft. per sq.mi. At the mouth of Bijou Creek on May 31, 1935, the graph shows a peak flow of 282,900 sec.-ft. from a total drainage area of 1,444 sq. mi., or 196 sec.-ft. per sq.mi. The approximate area over which the most intense precipitation occurred appeared to be 387 sq.mi. which would show a peak discharge of 731 sec.-ft. per square mile. Based upon a total runoff of 116,800 acre-ft., the runoff from the entire basin was 81 acre-ft. per sq.mi., and from the area covered by the most intense precipitation, 302 acre-ft. per sq. mi. While some of the results, particularly those of the Bijou Creek discharge, appear to be incredible and entirely beyond anything which has ever occurred in Colorado or adjacent areas, they are submitted for whatever value they may have.

The valleys of the streams herein mentioned were completely ravaged by the floods. The floods in the South Fork of the Republican and Arickaree rivers, destroyed either partially or in whole, 60 miles of railroad, many railroad and highway bridges, together with private and corporate property. The floods in Kiowa and Bijou creeks caused losses of hundreds of thousands of dollars to farming interests, highways, railroads and individuals. Two large flumes which carry the Bijou and Fort Morgan canals across Bijou Creek were destroyed and had to be rebuilt at a cost of $150,000. The Bijou ditch flume was replaced by a large inverted steel siphon. These facts are mentioned as indicative of the magnitude of the floods and tend to substantiate the tentative conclusions presented.

It is with great reluctance that the data with respect to the runoff in acre-feet is submitted, since the period covered by some of the floods could not be definitely ascertained. It is believed, however, that the determinations of the peak discharges of the streams are within reasonable limits of accuracy.

Letters to the Editor

More Levels and Maps Needed

Sir—I have read with a great deal of interest your editorials in the July 2, 1936, number of *Engineering News-Record* on "Points and Benchmarks" and "A Milestone."

As you know, the Coast and Geodetic Survey now has about 260,000 miles of accurately run lines of levels with permanent benchmarks set along each line. These benchmarks could be used as the starting points for any leveling that is done in connection with the establishment of benchmarks near buildings, which are to be investigated for stability.

As a matter of fact, in each city of the country there should be a network of accurate levels with the lines closely spaced in order that benchmarks might be available to engineers engaged on the many classes of work they undertake. There is hardly an engineering job that does not need accurate elevations. Those could be furnished by the city engineering department. I do not know of any city in the United States that is adequately covered with a network of precise leveling. The cost of the work would be very small. If one surveying party with about half a dozen men should be kept at work in a city for a year or two, there would be a tremendous amount of leveling done that I am sure would more than justify the cost involved in the work.

As for your editorial entitled "A Milestone," it seems to me that here the engineer and the geologist could come together to the advantage of both. There is no such thing really as stability of the earth. Through geological ages mountains have been built and others have been worn away. The movements necessarily are slow except during earthquakes when changes of elevation may be a foot or more. Geological literature is filled with references to changes in elevation of the earth's surface, and therefore we must conclude that for periods of time of a hundred, thousand or some other number of years, the earth's surface has not been stable. Tidal observations and repetition of observations along lines of levels through a long period of time, some decades perhaps, should give a measure of the rate of change in elevation that will be taking place at any point along our coasts or in the interior of the country. In order that abundant data may be available at all places when engineering work is done, there should be good topographic maps, the control surveys for which furnish basic data for stability tests.

There is now a tremendous sentiment among engineers, planners, users of land and others in favor of high grade surveying and mapping. The National Resources Board, now the National Resources Committee, nearly two years ago proposed a national mapping plan, which if put into effect would have completed the topographic mapping of the country within ten years.

We are not a well mapped nation. While 47 per cent of our area is covered with topographic maps, not more than half those maps are satisfactory for modern use. It is quite certain that great economies would result in carrying on many of the activities of our people if topographic maps should be made ahead of engineering construction, planning and modifications in the present use of land. It is hoped that in the immediate future there may be a large increase in the amount of mapping done in this country.

From all accounts there is not so much unemployment among engineers now as there was a few years ago. However, there must be many recent engineering graduates who have not found engineering jobs. Many of those could be placed on geodetic and topographic surveying and thus have them employed on professional work. The maps made by them would add to the capital account of our nation.

WILLIAM BOWIE
Chief Division of Geodesy,
U. S. Coast and Geodetic Survey.

Washington, D. C.
July 23, 1936.

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Editor—F. E. Schmitt. Editorial Staff—V. T. Boughton, N. A. Bowers, W. G. Bowman, W. W. DeBerard, C. S. Hill, H. W. Richardson, E. J. Cleary

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New King's Highways

THE recent action of the Ministry of Transport in recommending the nationalization of an important mileage of the main roads of Great Britain is a striking reversal of British public-roads tradition. Even more than in the United States roadbuilding and upkeep has in Britain been the function of local government; counties and municipalities have controlled their road planning and practice at will and in no more than friendly cooperation with their neighbors. The result has been a through-road network formed by linking up neighborhood roads of various constructions. In recent years the Ministry of Transport has exercised a harmonizing function somewhat analogous to that of our federal Bureau of Public Roads, but not with controlling effect on old road routes and types. The result is that none of the trunk roads radiating from London to the ocean borders is of the same alignment, width or type for any considerable distance. It is now proposed to transfer 4,500 miles of these roads to national control to be developed into through routes of express-road structure and capacity. The action is a logical step to capitalize present possibilities in motor-vehicle speeds and capacities for highway transport.

Grass-Roots Publicity

A Publicity Campaign that has notable differences from usual practice has been developed by the Indiana highway department. Its distinctive purpose is to connect up the work of the department with the local interest of the individual road user. Its generously helpful agency is the local newspaper. Other means are also employed for broader audiences and to spread technical information on financing, economics and engineering, but the local newspaper is the means of access to the grass-roots interests that are wholly vital to department support. The interest of the newspaper is secured by the individual news service given it. News releases are issued weekly so that the news is news, but this is not all; the blanket story is followed by information for a purely local story. The editor is not merely told that $3,000,000 of contracts for road improvement have been awarded but he is told exactly by name and mileage the roads in his town or county that are to be improved. This specific information is the vital news for local featuring and will carry with it almost certainly all or some part of the general publicity story. Old as the procedure is, not many state highway departments have learned its effectiveness in building up public support. The lesson of Indiana is worth noting.

Drought and Water

Latest Field Data on the drought in the northern states indicate that in broad aspect it is a duplication of the 1934 visitation and aside from a few local variations will give little additional information on drought intensities. However, following so closely after the 1934 record it shows that a period of abnormally low rainfall may have long duration, which is likely to mean that estimates of safe water yield must be reduced. The fact that some cities are already forced to restrict water use testifies to the fact that not all supplies have been built up to an adequate level. Full provision for community requirements is particularly imperative because nothing now known gives ground for hoping that man will ever be able to combat drought effects over large regions. At least he has never succeeded in controlling rainfall. Further, in much of the region most exposed to the drought hazard, the westerly prairies and the plains area, enough reservoir storage to save the supply of wet years for the dry cycles is out of the question. It may be that exhaustive study of groundwater conditions will in time show larger resources to be available below the surface than are yet known, but failing this, water conservation in drought areas can hardly go beyond provision for drinking water supplies.

Zoning Enforcement

Enforcement of a Zoning Law after city officials have long winked at breaches and have let violators establish themselves is a serious municipal problem not easily solved. Milwaukee, generally considered one of the better-governed cities of the country, finds itself in this position now. A WPA survey has just turned in a list of 3,365 non-conforming use violations, more than 2,400 of which are for business invasions of residential zones. Not all are flagrant violations. Many establishments undoubtedly existed prior to the passage of the zoning ordinances sixteen years ago. Nevertheless officials are of the opinion that a majority of the violations are illegal and a further checkup on the WPA count is to be made. Just how the 600 groceries and 392 taverns got intrenched among residences is being asked with raised eyebrows. What to do with violators? Wholesale prosecutions seem too drastic, but wholesale legalization by change of the ordinance would be pernicious. It's a knotty problem; how Milwaukee extricates itself will be worth watching, particularly by the hundred and one other zoned cities equally lax in enforcement.

Through the Telescope

An Appeal for Broader Study than is often given to engineering problems was made at the Portland meeting of the American Society of Civil Engineers by R. H. Baldock, Oregon highway engineer, in setting forth the importance of a long outlook in highway planning. His premise was that in effect we design and build highways largely for traffic needs immediately visible, which usually means those of today and next year alone. Instead, since our highways must continue to serve through a long period of years, they should be fitted to the requirements of the far distant future wherever these can be discerned. Any failure to do so is certain to mean heavy losses. Mr. Baldock suggested that because the future is hard to foresee we are apt to take a shortsighted view, and that the engineer often looks at his design problems with a microscope where he ought to use a telescope for long perspective. There can hardly be any fundamental disagreement with this plea. Most of our existing roads reveal clearly that the engineer's prevision was outrun by traffic change within a short time after their completion: alinement, grades, width, curve minimum, banking, safeguarding and whatnot proved inadequate under the rapid evolution of traffic habits and vehicle design; costly obsolescence came about and extensive rebuilding or revamping was necessary. Quite naturally the engineer will hesitate to spend money on provision for future possibilities that may not come to pass, but often such provision will prove cheap insurance—in fact a really necessary safety margin.

Heat-Treated Bridge Wire

DEFINITE CONCLUSIONS as to the cause of the breaks in the heat-treated bridge wire of the original Mount Hope and Detroit suspension bridge cables are drawn by Messrs. Swanger and Wohlgemuth in their report of the Bureau of Standards' study of samples of the wire. These conclusions will not be universally accepted, nor is it desirable that they should be. The evidence marshalled in their behalf nevertheless is unusually persuasive, and engineers will want to familiarize themselves with it. The report is abstracted in this issue.

Separating the stated cause of failure into its component parts, the basic factor is seen to be the crystalline microstructure of the wire, which, unlike the fibrous structure of cold-drawn wire, is incapable of resisting the spread of cracks starting at surface irregularities. Coupled with this microstructure was a rigidity or stiffness which gave rise to the preforming operation, in which the wire was bent around a small sheave to give it a permanent set, so that it would theoretically lie dead against the curve of the anchor shoe. The resulting curvature was not true, which was inevitable under field conditions, and bending stresses were induced in the high-elastic-limit wire as soon as a pull was applied. A steady pull, according to the investigators, would not break the wire, but under fluctuating pull reversed bending stresses were set up, and the wire eventually snapped by fatigue as the surface cracks ate their way into the crystalline structure of the wire.

The exceptions so far entered against this conclusion do not destroy its validity so much as they supplement some of its component parts. The manufacture of the wire has been criticized. Faulty manufacture would be reflected in an increased brashness of structure and in surface defects; these would accentuate the spread of cracks and would merely make failure more rapid, not necessarily more certain. Rough handling during stringing would contribute to failure, but here again the resulting damage would only hasten failure, for tests showed that even extreme care in preforming the loops in the laboratory did not make the wire immune to breaks.

Preforming the wire stressed it to or beyond its yield point. What kind of loading will induce failure in wire so stressed? The investigators were able to break the wire only under reversed bending, which could be caused only by fluctuating loads. Slowly increasing tensile load, as would occur as the bridge deck was added or as wires progressively broke, unloading onto other wires, would not break the wires in the laboratory. Yet E. E. Thum, in a thoughtful critique of the report also published in this issue, holds to the belief that such creeping loads were a significant factor. With some reason, he questions the extreme unloading of the wire that the investigators found to be most damaging. Wind loads, the principal source of fluctuating stress, could hardly unload the wire completely. Mr. Thum, therefore, adheres to an original contention that "creep" of steel played a part in the failure.

The subject is complex enough to warrant continued reflection and discussion, particularly on the part of metallurgists and others who have long used similar heat-treated wire for various purposes. For bridge engineers, the inherent weaknesses obvious in a granular microstructure, and the absence of these weaknesses in the fibrous, scale-like form of cold-drawn wire, provide enough data for the rejection of heat-treated wire and the continued use of wire whose strength rests in a cold-drawing process. The bridgebuilding art will benefit from these costly failures and the careful investigation by the Bureau of Standards as it has benefited in the past when failures have temporarily retarded it.

Reform Still Distant

WHEREVER ONE GOES this year, east, west or south, activity in residential construction is the characteristic sign of the times. Small and large cities report housing shortages and an increased amount of building. Since most of the reports carry a note of surprise that shows the building revival to be unexpected, it appears that the numerous forecasts of the past two years based on the accumulated deficit in replacements during the depression went unheeded.

Much has been said and written of housing reform, of new methods of building, of lower costs and greater permanence. Does the new housing that is now being built show progress in these directions? Are improved types of house construction or sounder procedure on the part of developers putting an end to the unsubstantial and high-cost building that exploited the homeseeker in the '20s? These questions inevitably come to mind in contemplating the revived housing activity.

No evidence of any marked change can be noted. Current housebuilding uses the old materials and follows the old methods in both construction and financing, with no essential improvement. Subdividers and group builders operate as formerly. Material and labor costs are not perceptibly lower, but on the contrary there are complaints of shortage in building-craft workers that point toward even higher building prices in 1937 and '38. Finally, building finance and real estate marketing are as unsatisfactory as before.

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August 13, 1936

New King's Highways

THE recent action of the Ministry of Transport in recommending the nationalization of an important mileage of the main roads of Great Britain is a striking reversal of British public-roads tradition. Even more than in the United States roadbuilding and upkeep has in Britain been the function of local government; counties and municipalities have controlled their road planning and practice at will and in no more than friendly cooperation with their neighbors. The result has been a through-road network formed by linking up neighborhood roads of various constructions. In recent years the Ministry of Transport has exercised a harmonizing function somewhat analogous to that of our federal Bureau of Public Roads, but not with controlling effect on old road routes and types. The result is that none of the trunk roads radiating from London to the ocean borders is of the same alignment, width or type for any considerable distance. It is now proposed to transfer 4,500 miles of these roads to national control to be developed into through routes of express-road structure and capacity. The action is a logical step to capitalize present possibilities in motor-vehicle speeds and capacities for highway transport.

Grass-Roots Publicity

A PUBLICITY CAMPAIGN that has notable differences from usual practice has been developed by the Indiana highway department. Its distinctive purpose is to connect up the work of the department with the local interest of the individual road user. Its generously helpful agency is the local newspaper. Other means are also employed for broader audiences and to spread technical information on financing, economics and engineering, but the local newspaper is the means of access to the grass-roots interests that are wholly vital to department support. The interest of the newspaper is secured by the individual news service given it. News releases are issued weekly so that the news is news, but this is not all; the blanket story is followed by information for a purely local story. The editor is not merely told that $3,000,000 of contracts for road improvement have been awarded but he is told exactly by name and mileage the roads in his town or county that are to be improved. This specific information is the vital news for local featuring and will carry with it almost certainly all or some part of the general publicity story. Old as the procedure is, not many state highway departments have learned its effectiveness in building up public support. The lesson of Indiana is worth noting.

Drought and Water

LATEST FIELD DATA on the drought in the northern states indicate that in broad aspect it is a duplication of the 1934 visitation and aside from a few local variations will give little additional information on drought intensities. However, following so closely after the 1934 record it shows that a period of abnormally low rainfall may have long duration, which is likely to mean that estimates of safe water yield must be reduced. The fact that some cities are already forced to restrict water use testifies to the fact that not all supplies have been built up to an adequate level. Full provision for community requirements is particularly imperative because nothing yet known gives ground for hoping that man will ever be able to combat drought effects over large regions. At least he has never succeeded in controlling rainfall. Further, in much of the region most exposed to the drought hazard, the westerly prairies and the plains areas, enough reservoir storage to save the supply of wet years for the dry cycles is out of the question. It may be that exhaustive study of groundwater conditions will in time show larger resources to be available below the surface than are yet known, but failing this, water conservation in drought areas can hardly go beyond provision for the drinking water supplies.

Zoning Enforcement

ENFORCEMENT OF A ZONING LAW after city officials have long winked at breaches and have let violators establish themselves is a serious municipal problem not easily solved. Milwaukee, generally considered one of the better-governed cities of the country, finds itself in that position now. A WPA survey has just turned in a list of 3,365 non-conforming use violations, more than 2,400 of which are for business invasions of residential zones. Not all are flagrant violations. Many establishments undoubtedly existed prior to the passage of the zoning ordinances sixteen years ago. Nevertheless officials are of the opinion that a majority of the violations are illegal and a further checkup on the WPA count is to be made. Just how the 600 groceries and 392 taverns got intrenched among residences is being asked with raised eyebrows. What to do with violators? Wholesale prosecutions seem too drastic, but wholesale legalization by change of the ordinance would be pernicious. It's a knotty problem; how Milwaukee extricates itself will be worth watching, particularly by the hundred and one other zoned cities equally lax in enforcement.

Through the Telescope

AN APPEAL FOR BROADER STUDY than is often given to engineering problems was made at the Portland meeting of the American Society of Civil Engineers by R. H. Baldock, Oregon highway engineer, in setting forth the importance of a long outlook in highway planning. His premise was that in effect we design and build highways largely for traffic needs immediately visible, which usually means those of today and next year alone. Instead, since our highways must continue to serve through a long period of years, they should be fitted to the requirements of the far distant future wherever these can be discerned. Any failure to do so is certain to mean heavy losses. Mr. Baldock suggested that because the future is hard to foresee we are apt to take a shortsighted view, and that the engineer often looks at his design problems with a microscope where he ought to use a telescope for long perspective. There can hardly be any fundamental disagreement with this plea. Most of our existing roads reveal clearly that the engineer's prevision was outrun by traffic change within a short time after their completion; alinement, grades, width, curve minimum, banking, safeguarding and whatnot proved inadequate under the rapid evolution of traffic habits and vehicle design; costly obsolescence came about and extensive rebuilding or revamping was necessary. Quite naturally the engineer will hesitate to spend money on provision for future possibilities that may not come to pass, but often such provision will prove cheap insurance—in fact a really necessary safety margin.

Heat-Treated Bridge Wire

DEFINITE CONCLUSIONS as to the cause of the breaks in the heat-treated bridge wire of the original Mount Hope and Detroit suspension bridge cables are drawn by Messrs. Swanger and Wohlgemuth in their report of the Bureau of Standards' study of samples of the wire. These conclusions will not be universally accepted, nor is it desirable that they should be. The evidence marshalled in their behalf nevertheless is unusually persuasive, and engineers will want to familiarize themselves with it. The report is abstracted in this issue.

Separating the stated cause of failure into its component parts, the basic factor is seen to be the crystalline microstructure of the wire, which, unlike the fibrous structure of cold-drawn wire, is incapable of resisting the spread of cracks starting at surface irregularities. Coupled with this microstructure was a rigidity or stiffness which gave rise to the preforming operation, in which the wire was bent around a small sheave to give it a permanent set, so that it would theoretically lie dead against the curve of the anchor shoe. The resulting curvature was not true, which was inevitable under field conditions, and bending stresses were induced in the high-elastic-limit wire as soon as a pull was applied. A steady pull, according to the investigators, would not break the wire, but under fluctuating pull reversed bending stresses were set up, and the wire eventually snapped by fatigue as the surface cracks ate their way into the crystalline structure of the wire.

The exceptions so far entered against this conclusion do not destroy its validity so much as they supplement some of its component parts. The manufacture of the wire has been criticized. Faulty manufacture would be reflected in an increased brashness of structure and in surface defects; these would accentuate the spread of cracks and would merely make failure more rapid, not necessarily more certain. Rough handling during stringing would contribute to failure, but here again the resulting damage would only hasten failure, for tests showed that even extreme care in preforming the loops in the laboratory did not make the wire immune to breaks.

Preforming the wire stressed it to or beyond its yield point. What kind of loading will induce failure in wire so stressed? The investigators were able to break the wire only under reversed bending, which could be caused only by fluctuating loads. Slowly increasing tensile load, as would occur as the bridge deck was added or as wires progressively broke, unloading onto other wires, would not break the wires in the laboratory. Yet E. E. Thum, in a thoughtful critique of the report also published in this issue, holds to the belief that such creeping loads were a significant factor. With some reason, he questions the extreme unloading of the wire that the investigators found to be most damaging. Wind loads, the principal source of fluctuating stress, could hardly unload the wire completely. Mr. Thum, therefore, adheres to an original contention that "creep" of steel played a part in the failure.

The subject is complex enough to warrant continued reflection and discussion, particularly on the part of metallurgists and others who have long used similar heat-treated wire for various purposes. For bridge engineers, the inherent weaknesses obvious in a granular microstructure, and the absence of these weaknesses in the fibrous, scale-like form of cold-drawn wire, provide enough data for the rejection of heat-treated wire and the continued use of wire whose strength rests in a cold-drawing process. The bridgebuilding art will benefit from these costly failures and the careful investigation by the Bureau of Standards as it has benefited in the past when failures have temporarily retarded it.

Reform Still Distant

WHEREVER ONE GOES this year, east, west or south, activity in residential construction is the characteristic sign of the times. Small and large cities report housing shortages and an increased amount of building. Since most of the reports carry a note of surprise that shows the building revival to be unexpected, it appears that the numerous forecasts of the past two years based on the accumulated deficit in replacements during the depression went unheeded.

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CURRENT NEWS

Flood Control Program in Pennsylvania

Broad program of flood control legislation pending in Pennsylvania legislature; two bills have been passed

THE RECENTLY adjourned session of the Pennsylvania legislature had under consideration a broad program of legislation designed to set up flood control agencies for the state. Two bills have been passed and signed by the Governor; another awaits the gubernatorial signature. Final consideration of the other measures was prevented by the rush of relief legislation in the last days of the session, and they are held over until the 1937 session.

A bill, passed and signed by the Governor, appropriates $50,000 to meet the expenses of a survey which the Department of Forests and Waters is directed to make. The survey is to cover the various watersheds of the state and is intended to determine what flood control works are necessary and possible. A report and recommendation are to be prepared before February, 1937, to be submitted to the next session of the legislature.

The other bill on which final action was taken is an appropriation of $32,000 for flood control work along the Delaware River in Pike county.

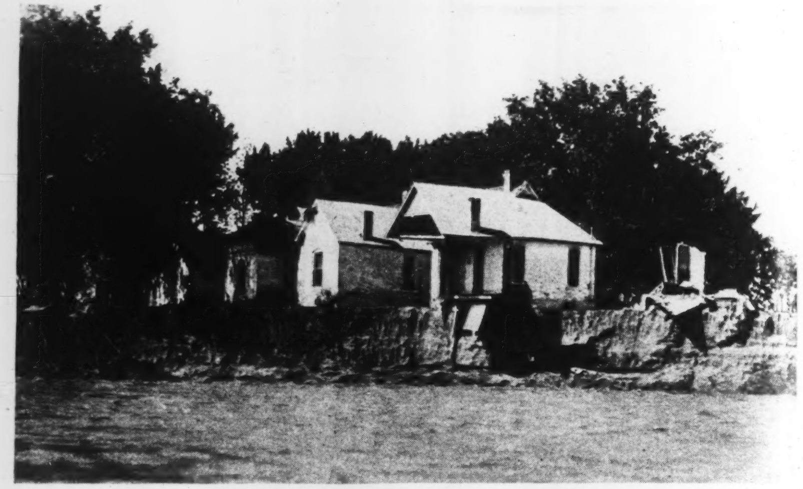
Provisions for local work

A bill has been passed by the legislature, but not yet signed by the Governor, which authorizes cities, boroughs, towns, and townships, separately or jointly, to construct flood control works, subject to the provision that such works do not interfere with the activities of existing state agencies.

A bill, passed by the House and now pending in the Senate, defines the powers and duties of the water and power resources board of the Department of Forests and Waters in relation to flood control. The board is empowered to set up flood control districts in the state and, in behalf of such districts, to exercise the power of eminent domain and to make assessments against property benefited in proportion to benefits. The board is also empowered to enter into compacts with other states for interstate flood control work. An appropriation of $2,000,000 is provided in the bill for such interstate works, and the water and power resources board is empowered to spend the money. Moreover, the board is made the agency of the state to receive money from and cooperate with the federal government in any flood control activities which may be undertaken by the government within the state. In this connection, the board is given the authority to exercise the same powers of eminent domain and assessment which it is given in behalf of the flood control districts. In negotiating with the federal government, the board may obligate the commonwealth to pay a portion of the cost of federal flood control work. Included in this bill is a provision for state operation of federal flood works.

An appropriation of $2,000,000 for flood control works within the state is still pending in the legislature.

Also pending is a bill directing the Department of Forests and Waters to investigate and report on the location, availability for purchase, and cost of submarginal land in the watersheds of the state, which may be purchased and used for reforestation for flood control purposes.



HOUSES SWEPT AWAY IN COLORADO FLOOD

The flooded Fountain River cutting the foundations from under a row of brick cottages, one of which has already collapsed. Before the water subsided, all the cottages had fallen into the river.

U. S. Engineers Will Appoint Eighteen Lieutenants

The War Department has announced that an examination will shortly be held for the appointment of eighteen new second lieutenants in the Corps of Engineers. This marks the first step to effect the increase in the officer strength of the organization which was authorized by the last session of Congress.

Applicants must be male citizens who will be between the ages of 21 and 30 on Feb. 1, 1937. The examination is open to any graduate of a technical college as well as to officers and enlisted men of the army and the National Guard. Applications for examination must be in the office of the corps area commander of the corps area in which the applicant is resident before Sept. 15, 1936.

Preliminary examinations will be held in October, and final examinations will begin Nov. 16, 1936. The examination, in addition to covering professional engineering subjects, will include a section covering general high school and college subjects. Honor graduates of certain selected colleges will be exempt from this last section. A rigid physical examination will also be given.

Applications for examination may be obtained from any corps area headquarters or from any military post.

Cloudburst Brings Flood In Colorado

On the morning of Aug. 6 a cloudburst occurred in the valley of the Fountain River about 25 miles north of Pueblo. Two inches of rain fell over an area of 25 square miles in about an hour. The region had been soaked with rains for the previous 12 days, and there was almost 100 per cent runoff.

The Fountain River rises in Ute Pass and has a length of about 60 miles. It joins the Arkansas River just below Pueblo. In the lower 42 miles of its length, it drops 1,240 ft. At periods of low flow, the river wanders around a wide valley, rising out of its bed in flood time.

Normal flow of the river at Pueblo is about 20 sec.-ft. At the height of the flood, the flow reached 15,000 sec.-ft. Flow figures for the days preceding the flood follow:

Date	Flow at Pueblo Sec.-Ft.
Aug. 2	10
Aug. 3	600
Aug. 4	800
Aug. 5	800
Aug. 6	15,000

Soil Mechanics Proceedings First Edition Sold Out

The first edition of 500 sets of the three-volume Proceedings of the International Conference on Soil Mechanics and Foundation Engineering has been sold out. It is intended to print a second edition if enough orders are received. Tentative orders for the second edition may be placed with The Treasurer, Foundation Conference, Graduate School of Engineering, Harvard University.

San Gabriel Suit Settled out of Court

The $2,600,000 recovery suit brought three years ago by the Los Angeles County Flood Control District against contractors on the original and abandoned San Gabriel Dam No. 1 (*EN-R*, June 1, 1933, p. 853), has been settled out of court for $737,986.

The case arose from the abandonment of the original San Gabriel Dam when the rock foundations of the site were found to be faulty. At that time the district paid an indemnity to the contractor in consideration of the abandonment of the contract. When, later, Sidney T. Graves, formerly supervisor of Los Angeles County, was convicted of accepting an $80,000 bribe from the contractor, for permitting the contractor to pursue the work in an improper manner, the flood control district brought suit to recover the amount of the indemnity and of all payments made to the contractor before abandonment of the dam. The defendants in the suit were Fisher, Ross, MacDonald and Kahn, Inc., the contractors; Sidney T. Graves; and E. T. Eaton, the former chief engineer of the flood control district. In the recent settlement, the amount of the indemnity was accepted in settlement of the entire claim.

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Readvertisement of Bids For Birmingham Housing Project

A second invitation for bids on the superstructure of the $2,500,000 slum clearance project in Birmingham, Ala., has been made. Bids will be opened on Sept. 13. No bids on this project were received at the previous advertisement. In the new advertisement, certain modifications in the bidding arrangements have been made, at the request of local contractors. The bidding period has been lengthened from 30 to 40 days. The time allowed for construction has been increased from 300 to 365 days. A greater number of plans and specifications will be made available to contractors than on the first request for bids, and specifications are being clarified on points where questions arose.

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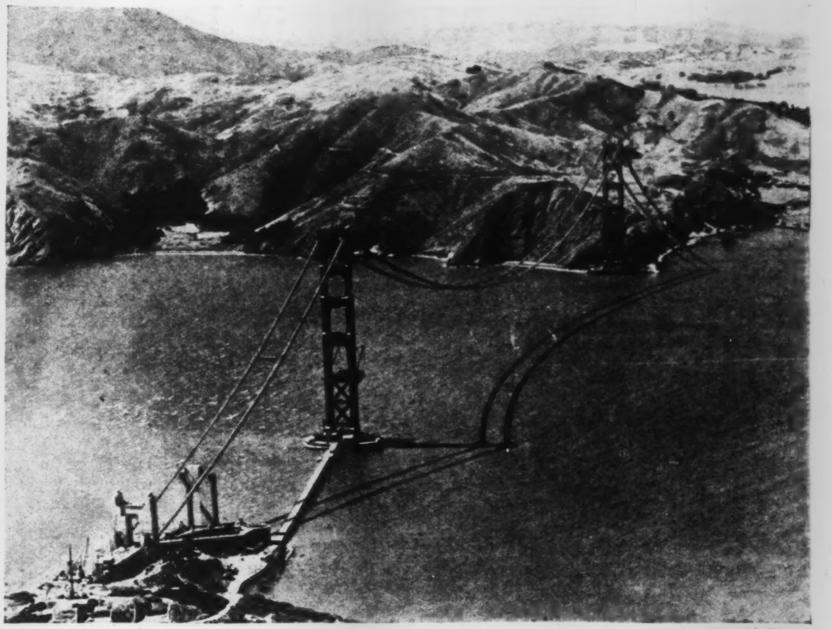
Thirteen Housing Contracts Let During July

The Housing Division of PWA has announced that during the month of July thirteen construction contracts were let with a value of $22,901,469. The jobs range in size from a $57,600 contract for foundations for a project in Toledo to the $4,979,670 superstructure contract for the Old Harbor Village project in Boston.

All 50 projects on PWA's housing program are now under construction, with one, Techwood, in Atlanta, completed. Since starting operations, the Housing Division has let 91 construction contracts with a total value of $48,199,790.

The contracts let during July were for the foundations of projects in Todelo, Ohio, and Omaha, Nebr., and for the superstructures of projects in Jacksonville, New York City, Boston, Buffalo, Dallas, Memphis, Columbia, S. C., Chicago, Louisville, Nashville and Oklahoma City.



Wide World Photo

CABLE SPINNING COMPLETED ON GOLDEN GATE BRIDGE

Spinning of the cables of Golden Gate Bridge has been completed. The next step is the installation of cable bands and hangers and wrapping the cables. A few panels of the stiffening truss have been built out toward the channel from the near tower.

Plans Approved For East River Tunnel

Plans for the proposed Midtown vehicular tunnel to connect the boroughs of Manhattan and Queens have been approved by the New York City Tunnel Authority. The plans have been submitted to the War Department for approval.

Full details of the design have not yet been released, but are expected to be made public within the next few days.

The first bids on the project, for a construction shaft in Queens, are expected to be advertised on Aug. 15 and opened about Sept. 1. It is planned to begin work Sept. 15.

The tunnel will be financed by PWA funds, a grant of $11,235,000 and a loan of $47,130,000.

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Vancouver Suspension Bridge Approved by Government

Approval has been granted by the Canadian government to plans for the First Narrows bridge across the entrance to the harbor at Vancouver, B. C. The bridge, estimated at about $6,000,000, is to be built by British Pacific Properties, Ltd. Its suspension span is to be 1,500 ft. long with a 200-ft. vertical clearance. The two approach spans are each to be 575 ft. long. A 2,900-ft. viaduct will serve as approach to the north end. The bridge will span the First Narrows near Prospect Point.

W. G. Swan, Vancouver, is the local engineer for the bridge company; Monsarrat and Pratley, Montreal, are designing and supervising engineers and Robinson and Steinman, New York, are consultants. Bids are to be called late in the fall.

SOCIETY CALENDAR

CANADIAN GOOD ROADS ASSOCIATION, annual meeting, Charlottetown, P.E.I., September 1-3.

THIRD WORLD POWER CONFERENCE, Washington, D. C., Sept. 7-12.

PUBLIC WORKS CONGRESS, joint meeting, American Society of Municipal Engineers and International Association of Public Works Officials, Toronto, Ont., September 28-30.

CANADIAN INSTITUTE OF SEWAGE AND SANITATION, Royal York Hotel, Toronto, Oct. 1-2, 1936.

INTERNATIONAL CONFERENCE ON STEEL CONSTRUCTION, Berlin, Germany, Oct. 2-7.

INTERNATIONAL ASSOCIATION FOR BRIDGE AND STRUCTURAL ENGINEERING, second congress, Berlin, Germany, Oct. 11.

NATIONAL SOCIETY OF PROFESSIONAL ENGINEERS, annual convention, Washington, D. C., Oct. 12-13.

AMERICAN WATER WORKS ASSOCIATION, Central States Section, Cleveland, Ohio, Aug. 19-21.

ENGINEERING INSTITUTE OF CANADA, maritime general professional meeting, St. Johns, N. B., Aug. 27-29.

AMERICAN WATER WORKS ASSOCIATION, Rocky Mountain Section, Denver, Colo., Sept. 21-23.

NEW ENGLAND WATER WORKS ASSOCIATION, annual convention, New York, N. Y., Sept. 22-25.

AMERICAN WATER WORKS ASSOCIATION, Southwest Section, section meeting, Fort Smith, Ark., Oct. 12-13.

AMERICAN WATER WORKS ASSOCIATION, New York Section, fall meeting, Saratoga Springs, N. Y., Oct. 22-23, 1936.

EXAMINATIONS for registration as Engineers, Land Surveyors and Architects will be held as follows:

NEW MEXICO, examination for professional engineers and land surveyors, Santa Fe, Sept. 12.

Driest Four Months in History

Washington Correspondence

THE DROUGHT is reported now to be the most severe in the climate history of the United States by J. B. Kinser, of the Weather Bureau. July was the fourth consecutive month in which little rain fell in the interior valleys and the Great Plains. This four-month period has been the driest of record in the Dakotas, Minnesota, Wisconsin, Iowa, Missouri, Illinois, and Indiana, and the second driest in Ohio, Kentucky, Oklahoma, Kansas, and Montana.

The condition of surface and ground water supplies in 13 states is indicated in reports received during the past week from district engineers of the Geological Survey.

Arkansas

Although water is becoming scarce in many sections of Arkansas, streams are better than in 1934. Wells, on the other hand, are lower in some sections than ever before. Stream flow in second feet for the week ending Aug. 1 as compared with the same date in 1934 is as follows:

River	1936 Sec.-ft.	1934 Sec.-ft.
White	190	178
Red	1,970	814
Little	21	4

Iowa

In Iowa the stream flow still is above the minimum of 1934. However, unless heavy rains occur, new minima will be reached on many of the streams in the state. On July 30 flows were reported as follows:

Station	July 30, 1936 Sec.-ft.	1934 Sec.-ft.
Mississippi River at LaClaire	13,300	10,500
Turkey River at Garber	100	46
Iowa River at Wapaho	700	525
Racoon River at Vanmeter	40	18
Des Moines River at Boone	70	51

River freight traffic on the Mississippi River above St. Louis is now practically at a standstill.

In a group of observation wells in western Iowa, the average water level on July 27 was more than 3 ft. higher than in the middle of July, 1934. Since the 1934 drought, the water levels in these wells have risen as a result of heavy rains, until at the end of May, 1936, the average level was about 7 ft. higher than at the lowest stage in the fall of 1934. Since the end of May, however, the water levels have declined an average of nearly 2½ ft.

Montana

There was no improvement in the drought situation in eastern Montana and western North Dakota during the week ending Aug. 1. There has been no precipitation within the area, and the temperature has been well above normal. The drought area may be considered as extending west and northwest to the base of the Rocky Mountains. The area near the headwaters of the Milk and Marias rivers is now short of water for irrigation and domestic use. The flow of all streams is extremely low, and a number of the smaller tributaries of the Milk River are completely dry. Reports received from most of the large cities and towns in the area indicate that the water supply is decreasing and that in many cases the use of water has been restricted.

Kansas

The heat wave that has existed in Kansas during most of June and July has been broken, and rains have fallen over a large part of the State. Although these rains have been inadequate to replenish the water supplies appreciably, they have at least stopped the rapid diminution of river discharge.

In a group of observation wells near Mankato, in north-central Kansas, the average water level on July 31 stood almost a foot higher than the average on the same date in 1934, although the water levels in some of the wells stood lower.

Minnesota

Drought conditions in Minnesota are unbroken, but temperatures have lowered considerably. Despite the few scattered showers that have occurred, the streams have continued to fall daily. Municipal water supplies in the Red River basin are very critical, and a large number of small fires have occurred in the grass areas and peat swamps.

Nebraska

The water levels in the wells in most parts of Nebraska are lower than they were at the corresponding period last year, and above the levels at the corresponding period in July, 1934. The water levels in wells in northeastern Nebraska, are however, as low or lower than at the same time two years ago, with many new low stages, but only relatively few wells are reported to be dry. Wells in southeastern Nebraska, where acute water shortage existed in 1934, are appreciably above the stage of that year. Ground water levels in the Sand Hill area are lower than in 1934. Water levels in the Platte River valley are generally lower than in 1935 but higher than in 1934.

During the week ending July 25 the flow of streams in Nebraska as compared with the same week in 1934 for the following streams are reported:

River	1936 Sec.-ft.	1934 Sec.-ft.
Niobrara River near Spencer, Neb.	526	550
Loup River at Columbus, Neb.	1,020	724
North Fork Republican River at Colorado-Nebraska line	5.5	5

New York

The records of stream flow obtained at Geological Survey river measurement stations show that the surface water supplies in part of New York increased somewhat during the week ending Aug. 1.

However, in some areas the flow decreased during the week. The flow of the Genesee River in Letchworth State Park decreased from 63 sec.-ft. on July 24 to less than 50 sec.-ft. on July 31. Similarly the flow of the Susquehanna and Oswego rivers dropped appreciably during the same period. In the Oswego basin many of the mills are now threatened with shut-down because of insufficient water. The combined drainage areas of the Oswego and Susquehanna basins cover about 35 per cent of the area of the State.

Despite moderate temperatures which have assisted materially in preserving ground water supplies, the ground water levels in many parts of the state fell considerably during the week.

Ohio

The surface flows in Ohio have almost reached the minimum flows of 1934 but are still somewhat higher than those which occurred during the drought of 1930. Water shortages have been reported in eleven municipalities, and there is considerable shortage for farm use.

Oklahoma

In a group of observation wells near Stillwater, in central Oklahoma, the average water level on July 27 of this year was about 1.5 ft. lower than at the end of July, 1934, owing to a general lack of substantial underground replenishment since the 1934 drought.

Texas

In a group of observation wells, near Temple, in the central part of Texas, the water levels rose considerably in May of this year, in response to rainfall, and on July 29 the average level was about 1.5 ft. higher than at the same time in 1934, although lower than at the same time in 1935.

Wyoming

During the week ending July 25 the flow of streams in Wyoming continued to decrease, and, as indicated by the records of discharge, are now practically the same as during the same period of the 1934 drought:

River	1936 Sec. ft.	1934 Sec.-ft.
Tongue River near Decker, Mont. (near Wyoming State line)	65	61
Goose Creek near Sheridan, Wyo.	40	33
Powder at Arvada, Wyo.	71	88

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Detroit to Hire Engineers

The Detroit Civil Service Commission will hold examinations in August and September, for civil engineers to work on the Detroit sewage disposal project. The positions to be filled are: senior civil engineering draftsman, twenty vacancies; junior civil engineer (sanitary design), ten vacancies; junior assistant civil engineer (sanitary design), six vacancies; assistant civil engineer (sanitary design), three vacancies; senior structural engineering draftsman, sixteen vacancies; junior structural engineer (design), eight vacancies. These positions are to be held for the duration of the sewage disposal project.

Examinations will be held twice, once in August in Detroit only, and again, on Sept. 10, in 25 cities. There is a general requirement in all examinations that applicants must have education equivalent to graduation from a university or engineering school and suitable experience commensurate with the demands of the position.

G. H. Walbridge Dies at 66

George Hicks Walbridge died on Aug. 5 in New York City at the age of 66. He was an authority on hydro-electric construction. Mr. Walbridge received a degree in civil engineering from the Cornell School of Engineering in 1890. Immediately after his graduation, Mr. Walbridge became associated with the White-Crosby Co., which shortly thereafter became J. G. White & Co., as vice-president and manager. He directed the construction of the first high tension transmission line from Niagara Falls to Buffalo and of the electric rapid transit systems in Buffalo, Baltimore, and other cities. In 1905 Mr. Walbridge began construction of hydro-electric plants for the Central Colorado Power Co., of which he became president in 1907. In 1910 he became president of the Arizona Power Co. Prior to his partial retirement of the last ten years, Mr. Walbridge was actively concerned in the management of many industrial, utility and financial organizations.

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Brief News

The Regulation Requiring the use of 90 per cent relief labor on the Sardis flood control reservoir in Mississippi has been changed to permit employment of available labor. The action was taken after it became apparent that the 90 per cent clause prevented the securing of a sufficient supply of labor.

The City of St. Louis has asked the United States Court of Appeals of the District of Columbia to dismiss the suit, now pending before it, seeking an injunction against construction of the $6,000,000 Jefferson Memorial project at St. Louis. The city asked dismissal of the suit on the grounds that, although it is an interested party, it was not included in the action. The request for an injunction, on the grounds that the Relief Act, under which PWA made an allotment to the project, is unconstitutional, was denied in the Supreme Court of the District of Columbia and is now pending before the higher court.

Ten Bids Were Received for furnishing and installing electrical equipment for the Midtown Hudson tunnel, on Aug. 6, by the Port of New York Authority. The bids ranged from $948,367 to $1,420,000. J. Livingston & Co. of New York City was low. The work to be done includes furnishing and installation of the lighting system, telephone system, traffic control and fire alarm system, and the supervisory control system for the power supply. These are to be installed in the south tube and the ventilation buildings, all of which are still under construction. It is expected that the work under this contract will be completed in September of 1937.

The Willacy County Irrigation District No. 1 has entered into a contract with the Mercedes-Weslaco, Cameron and Hidalgo Counties Irrigation District No. 8 by which the Willacy County district will not build its own pumping plant on the Rio Grande, but will make use of the existing pumping plant of the Mercedes-Weslaco, Cameron and Hidalgo Counties District. The latter district will be supplied with funds to rebuild and enlarge the plant and to improve its main canal for the joint use of both districts. The Willacy County district will build an intersecting canal from a point near Mercedes to connect with the 75,000 acre Willacy irrigation project.

The Dominion Government of Canada has, to date, expended $14,551,133 on the construction of the trans-Canada highway, in grants to provinces. All the provinces have had some grants, but Ontario has received the main part of the total, amounting to $10,239,072.

The City of Little Rock, Ark., has awarded the contract for a 33-mile, 39-in. pipeline, from Saline River to the city, to the Lock Joint Pipe Co. of Ampere, N. J., on a bid of $1,561,682. Bids will be opened later for the building of a dam, at an estimated cost of $1,178,000. When the work is done, the city will have an entirely new water supply and will be the last city in the state to quit the Arkansas River as a source of water.

The Public Service Commission of Pennsylvania has authorized the Delaware River Tunnel Corp. to construct the proposed $15,000,000 tunnel under the Delaware River between Gloucester County, New Jersey, and Hog Island, Pa. The proposed tunnel has already been approved by the Secretary of War. The tunnel will be 1,500 ft. long, and will be 53 ft. below mean low water. No funds to start construction are available but it is hoped that financing can be arranged through the RFC or the PWA.

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Personals

Sherman Smith, assistant professor of civil engineering at the University of Toledo, has been named city traffic engineer of Toledo.

A. A. Cross, district engineer in the maintenance department of the New Haven R.R., has been appointed office assistant, with headquarters at New Haven.

John G. Sillenbeck has been appointed chief PWA engineer for Rhode Island. He has been acting in that capacity since November, 1935.

Gilbert H. Friend has been appointed chief of the U. S. Engineers on the construction of the Muskingum watershed conservancy district project.

A. M. Dow, director of operations for the WPA in Northwest Missouri, has been transferred to the state WPA offices to handle a newly created department of engineering statistics.

Edwin W. Burritt has resigned his position as state engineer of Wyoming to serve as special engineering counsel for Wyoming in the North Platte River litigation.

J. Buckley Bryan has resigned his position as district director of the WPA in Westchester County, N. Y., to accept an appointment as engineering aide to the New York State Director of the Budget.

Charles F. Looney, who has been taking graduate work in civil engineering at the University of Illinois, has been appointed as the university's representative on the construction of the new Medical and Dental College of the University of Illinois.

C. H. Cutler has resigned his position as chief engineer of the Missouri state highway department to accept a position as chief engineer of the Kentucky state highway department.

Carl Brown has been made chief highway engineer of the Missouri state highway department. Mr. Brown was formerly assistant chief highway engineer.

John R. Haynes has been re-elected president of the Los Angeles, Calif., board of water and power commissioners. He has been a member of the board for fifteen years, and has served three years as president.

J. J. Corbett, division engineer at Kansas City for the Missouri state highway department, has been promoted to construction engineer. He will be succeeded at Kansas City by W. H. Burgwin, assistant construction engineer for the highway department.

Gus E. Hauser has resigned his position as assistant city engineer of Memphis to join the S. & W. Construction Co. Mr. Hauser will be succeeded by R. L. Carpenter, previously resident engineer at Memphis for the Tennessee State Highway Department.

John F. Gilchrist, formerly chairman of the New York City Rapid Transit Commission, has been appointed special deputy commissioner of the New York State Bureau of Motor Vehicles. Mr. Gilchrist will conduct a special survey of highway traffic problems and formulate plans for a new system of licensing motor vehicle operators.

Prof. F. A. Russell has been appointed chairman of an executive committee to function as head of the school of engineering and architecture of the University of Kansas, pending the selection of a successor to the late Dean George C. Shaad who died unexpectedly while attending the American Institute of Electrical Engineers Convention in Pasadena, Calif., the latter part of June. Other members of the committee are Professors W. C. McNown, J. O. Jones and E. D. Hay.

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Obituary

George T. Hammond, who, before his retirement in 1933, was an assistant engineer for the New York City sewer department, died on Aug. 9 in Cranston, R. I., at the age of 73.

Ray Whitenack, chief engineer of the Oklahoma relief administration, died in Oklahoma City on July 31.

Vincent U. Ferri, a retired civil engineer formerly on the staff of the New York City Board of Transportation, died Aug. 4 in Brooklyn.

William B. Causey died in Chicago on Aug. 10 at the age of 71. Since 1925 Mr. Causey has been associated with the White Construction Co. of Chicago. He served for several years as city manager of Norfolk, Va. After the World War he served as a member of the American Relief Mission, stationed in Austria, and as technical adviser to that government.

F. C. Krotter, 68, a Nebraska road contractor, died recently in Palisade, Neb.

Claude William Kinder, for 31 years general manager and engineer in chief of the Imperial Chinese Railways, died in London on Aug. 9 at the age of 84.

Irvin F. Lehman, president and one of the founders of the Blaw-Knox Co., died Aug. 5 in Hartford, Conn., at the age of 59. Mr. Lehman was a director of many steel, banking, and industrial concerns.

Frederick E. Schall, consulting bridge engineer for the Lehigh Valley Railroad Co., died on Aug. 6.

William E. Sheen, formerly superintendent of streets at Peabody, Mass., died recently at the age of 89.

Robert N. Navel, 71, a building contractor in Buffalo, N. Y., died in that city on Aug. 3.

Frederich H. Andrews, 52, a construction engineer attached to the U. S. Treasury Department, died in Boston Aug. 6. Mr. Andrews was formerly a member of the engineering staff of the United Fruit Co.

Roy Huffelman, consulting engineer for the Chattanooga Electric Power Board, died in Memphis, Tenn., on Aug. 6.

John A. Kerby, a retired architect of New York City, died on Aug. 4 at the age of 78.

John C. Heidrich, a building contractor in Albany, N. Y., for the past 25 years, died there on Aug. 6.

William Crawford, 58, a building contractor in New York City, died in Southampton, L. I., on Aug. 10.

CONSTRUCTION STATISTICS FOR THE WEEK

Engineering awards for the week, at $41,282,000, are lower than last week's volume of $53,253,000 and below the average week to date. This total is made up of private, $9,377,000 and public, $31,905,000, of which federal is $12,410,000 and state and municipal, $19,495,000. Corresponding values for a year ago are: total, $45,028,000; private, $6,870,000; public, $38,158,000; federal, $24,886,000; and state and municipal, $13,272,000.

Sewerage, industrial buildings, earthwork, drainage, and waterways awards are higher this week, but a large drop is registered in both commercial and public buildings. The classification totals are: industrial buildings, $5,294,000; commercial buildings, $2,850,000; public buildings, $4,307,000; highways, $9,723,000; bridges, $3,045,000; sewerage, $3,382,000; waterworks, $677,000; earthwork, drainage, $11,135,000; unclassified, $869,000.

The larger awards for the week include: warehouse, J. E. Seagram & Sons, Lawrenceburg, Ind., $800,000; Meadowbrook Residences Development, Plainfield, N. J., $500,000; apartment houses, Ardsley-on-Hudson, N. Y., $1,000,000; airport terminal, Contract 7, San Francisco, Calif., $677,000; highways, Missouri, $954,000; Nebraska, $1,450.000; North Carolina, $562,000; Wisconsin, $598,000; grade separation, Chicago and North Western Railway, Milwaukee, Wis., $443,000; sanitary sewer collecting system, Corpus Christi, Tex., $603,000; sewerage system, Oshkosh, Wis., $448,000; Sections 11 and 12, Detroit River Interceptor, Detroit, Mich., $1,805,000; New LaGrange Lock and Dam, Beardstown, Ill., $1,965,000; Los Angeles-Long Beach Breakwater Project 773, U. S. Engineers, Los Angeles, Calif., $2,145,000; excavating and disposing of 54,000 cu.yd. overburden and 415,000 cu.yd. ledge rock in New York and New Jersey Channel Section 1, U. S. Engineers, $1,959,000; and Sections 2 and 3, $2,038,000; dredging, Section 6, channel north of Shooters Island, New York and New Jersey, U. S. Engineers, $636,000; dredging, Sections A and B from Chocktawachee Bay to West Bay between Pensacola and Mobile, U. S. Engineers, Florida, $721,000 and $627,000.

New capital for the week comprises corporate security issues, $660,000 and state and municipal bond sales, $5,791,000.

CONTRACTS

(Thousands of Dollars)

	Weekly Average August 1935	Weekly Average Prev. 4 Weeks	Week Aug. 13 1936
Federal Government	$12,611	$12,031	$12,410
State and Municipal	12,396	20,038	19,495
Total public	$25,007	$32,069	$31,905
Total private	6,439	13,315	9,377
Week's total	$31,446	$45,384	$41,282

Cumulative to date:
1935..$845,822,000 1936..$1,442,505,000

NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

1936	Week Aug. 13	Cumulative
State and municipal...	$5,791	$318,195
PWA non-federal	....	56,233
RFC loans	....	12,479
Corporate issues	660	151,647
PWA loans, Private...	....	*—1,665
Total Non-Federal...	$6,451	$536,889
Federal	...	691,237
Total new capital....	$6,451	$1,228,126

Cumulative to date:
1935..$1,292,268,000 1936..$1,228,126,000

*Bond sales in this classification exceed reallotments during current year.

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities.

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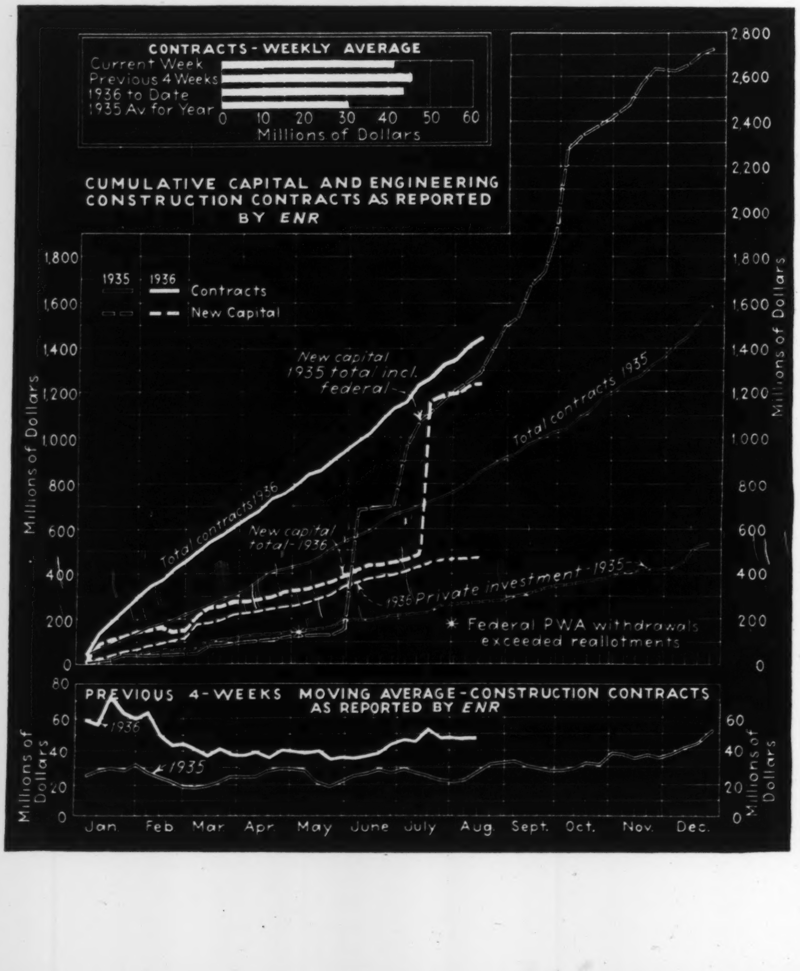ENR Cost	1913 =100	1926 =100	ENR Volume	1913 =100	1926 =100
Aug., 1936	**208.10**	**100.03**	**July, 1936**	**188**	**82**
July, 1936	204.40	98.25	June, 1936	194	85
Aug., 1935	195.10	93.82	July, 1935	96	42
1935 (Av.)	195.22	93.84	1935 (Av.)	135	58
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47